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Contents

Foreword

Management of freeway traffic is an increasingly important issue as congestion levels increase and more and more freeways become subject to rebuilding. This collection of papers describes aids to improve management, beginning with Lindley's presentation of a computer-based approach to quantifying urban freeway congestion.

The next four papers relate to problems arising from reconstruction under traffic in urban areas. Cohen and Clark show how traffic simulation models may be used to analyze alternatives for handling traffic during reconstruction, using as an example the INTRAS model on a bridge project in Washington, D.C. Leonard and Recker also report the use of models in choosing alternative construction and traffic management strategies in order to minimize associated direct and indirect losses. Reporting the user viewpoint, Bullard provides data on both Houston and Pittsburgh projects and how commuters perceived and responded to the impacts created by reconstruction. Janson et al. survey and review reconstruction techniques, traffic accommodation strategies, quality control, and project development processes that have been applied to mitigate travel impacts.

Alpern and Gersten report on the effectiveness of a freeway simulation model to evaluate an exclusive bus-high-occupancy-vehicle lane in northern New Jersey and needed additional procedures required to complete the study. Van Aerde et al. present the results of their evaluation of 14 models initially chosen for application in Ontario's freeway corridors, and detail their conclusions as to the most promising candidates. Dillon and Hall, using data from Ontario also, analyze the applicability of catastrophe theory to improved understanding of freeway operations. A new method of analyzing delay is presented by Derr for use in estimating the benefits of alternative projects for freeway corridor surveillance, communication, and control systems.

Chen and May address a fundamental question in traffic management—the reliability of traffic detectors—and offer a diagnostic test and software algorithm to offset detector deficiencies. The last paper, by May, updating an earlier paper and assessing freeway simulation model developments in the 1980s, also provides an extensive list of literature references.

A Methodology for Quantifying Urban Freeway Congestion

JEFFREY A. LINDLEY

Urban freeway congestion is a serious and growing national problem, one that is receiving increasing attention from transportation engineers, planners, and researchers as well as local, state, and national officials. When attempting to quantify this problem or evaluate alternative solutions for a single freeway or for an urban area, one finds that a convenient methodology to calculate urban freeway congestion parameters such as delay, excess fuel consumption, and user costs does not exist. A computerized methodology is described that was developed to quantify urban freeway congestion parameters on a national basis. This methodology was applied to a national computerized database, but could be easily used by local agencies because the required input data are minimal. The procedure can be applied to a single freeway segment or to several segments in an urban area. The methodology described in this paper forms the basis of a user-friendly microcomputer program for calculating urban freeway congestion.

Urban congestion is a serious and growing national problem, one that is receiving increasing attention from transportation engineers, planners, and researchers as well as local, state, and national officials. These professionals typically must evaluate several types of improvements for alleviating congestion on urban freeways, including widening, surveillance and control systems, ramp metering, incident management, motorist information systems, high-occupancy-vehicle (HOV) facilities, and low-cost geometric improvements. When trade-off analyses for these improvements are performed, a simplified methodology for calculating urban freeway congestion parameters would be useful.

To quantify the national problem of urban freeway congestion for both existing and future traffic levels and to analyze the aggregate impacts of various methods of solving the problem, an FHWA staff research study was initiated. One of the outputs of the study was a consolidated computerized methodology to quantify urban freeway congestion parameters such as delay, excess fuel consumption, and user costs. This methodology is the subject of this paper.

DATA REQUIREMENTS

For the study on which this paper is based, the Highway Performance Monitoring System (HPMS) database maintained by FHWA was used. The HPMS database contains detailed geometric, traffic, and other data for approximately 50 percent of the urban freeway mileage in the nation. The HPMS data can be used to represent the total urban freeway system through the use of appropriate expansion factors supplied by each state.

The HPMS data items actually used are those that are typically readily available through local highway or traffic engineering agencies. The data items required by the methodology are as follows:

1. Section length,
2. Number of lanes,
3. Annual average daily traffic (AADT),
4. *K*-factor (percentage of AADT occurring during peak hour),
5. Peak-hour directional factor,
6. Shoulder width,
7. Lane width, and
8. Percent trucks.

The first six items are required to use the methodology. The last two are required only if an estimate of the freeway section capacity is desired. If this estimate is not desired, a value of directional freeway capacity must be specified as an input data item. (Any potential users of the methodology in this paper who intend to use HPMS data should carefully check the HPMS sampling basis for their particular state before developing data for individual urbanized areas. This is because some states have elected the option permitted under HPMS of sampling all urbanized areas within the state as a group.)

METHODOLOGY

Figure 1 is a flowchart of the steps in the analysis program developed to quantify urban freeway congestion parameters. The program is written in FORTRAN IV and was structured to handle the large data requirements of quantifying the urban freeway congestion problem on a national scale. Each of the major steps in the analysis program and how it was developed are given in the following paragraphs.

After the totals for the various calculated parameters have been initialized, the input data described in the previous section are read. The section capacity is then estimated if a directional-hourly capacity is not provided as an input data item. Capacity is calculated as follows:

$$C = 2{,}000NWT$$

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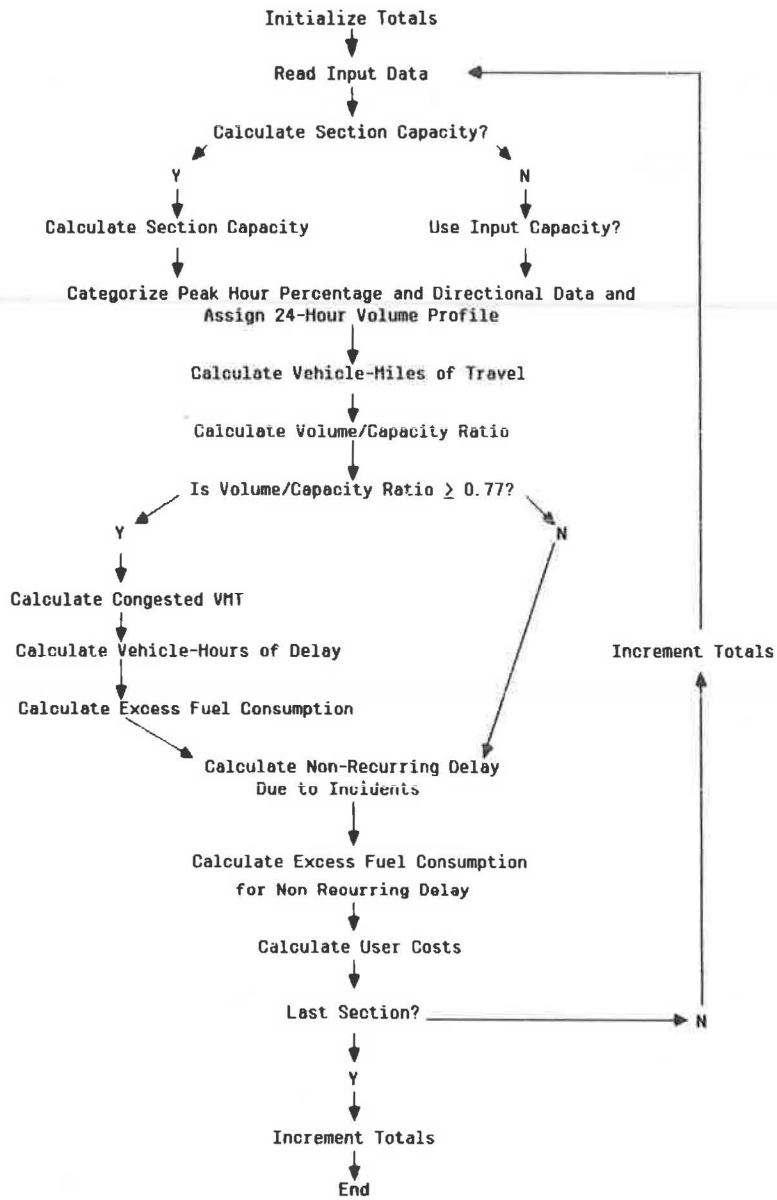

FIGURE 1 Steps in analysis program.

where

C = capacity,
N = number of lanes,
W = adjustment factor for lane width and lateral clearance, and
T = adjustment factor for truck presence based on percentage trucks and terrain.

Number of lanes is an input data item. The adjustment factor for lane width is approximated by using lane width and shoulder width (as a surrogate for distance to lateral obstructions) and the look-up tables in the 1985 *Highway Capacity Manual* (HCM) (*1*). The adjustment factor for trucks is also calculated by using the tables in the 1985 HCM. Percentage trucks is used directly and terrain is assumed to be basically level (two passenger-car equivalents per truck). Because the basic lane capacity in the foregoing equation is assumed to be 2,000 vehicles per lane per hour, design speed for the freeway is assumed to be at least 60 mph. The adjustment factor for population used in the 1985 HCM does not appear in this equation and is assumed to be 1.0 (commuter traffic) because the program applies only to urban freeways.

The next step in the analysis program is to assign a 24-hr volume profile to the freeway section. To simplify this process, several sets of traffic counts from I-66 and I-395 near Washington, D.C., were obtained. These counts, which were taken in 1983 and 1984 in various locations, represent a wide variety of peak-hour traffic percentages and directional factors. From them a total of twelve 24-hr volume profiles (expressed in terms of directional percentage of AADT) were developed for a "typical" urban freeway. The analysis program uses the input data for *K*-factor and directional factor to calculate a peak-hour directional percentage of AADT on the freeway section. An appropriate 24-hr volume profile is then selected on the basis of this percentage. The twelve 24-hr volume profiles developed are applicable for peak-hour directional percentages as low as 3.75 and as high as 9.25. The majority of urban freeway sections in the nation should fall into this range.

Total annual vehicle miles of travel is calculated next. This calculation is based entirely on input data. The equation for annual vehicle miles of travel for each section is given as follows:

$$VMT = AADT*LENGTH*365$$

where

VMT = total annual vehicle miles of travel,
$AADT$ = annual average daily traffic,
$LENGTH$ = expanded section length, and
365 = days per year.

Before the calculations for annual congested vehicle miles of travel are performed, it is necessary to select a point at which congested flow begins. For the purposes of this methodology, it was decided to qualitatively define congestion as operation of the freeway under conditions where a typical motorist's trip would be significantly delayed compared with the same trip under low-volume conditions. The numerical values selected to describe this point were taken from the 1985 HCM (*1*), Table 3-1 of which gives density, average travel speed, volume/capacity ratio (V/C), and maximum service flow values for various levels of service. The point selected to define congestion was the boundary between levels-of-service C and D. At this point, the density is approximately 30 passenger cars per lane mile, average travel speed is approximately 54 mph, V/C is approximately 0.77, and maximum service flow is approximately 1,550 passenger cars per lane per hour for 70-mph design speed facilities. The values of speed and V/C were the two key parameters used as decision values in the analysis program. It should be noted here that the threshold point for congestion chosen for this study, the boundary between levels-of-service C and D, is qualitatively the same as that used in the U.S. Department of Transportation's reports to Congress on the status of the nation's highways (*2*) and AASHTO's recommended design standard for urban freeways (*1*). The values were based on the 1965 *Highway Capacity Manual* (*3*). In the analysis program, V/C is calculated for the freeway section for each hour of a typical day. If V/C is greater than or equal to 0.77, the flow on the section is considered to be congested and the travel occurring on the section during the hour is considered congested travel.

The formula for calculating total annual congested vehicle miles of travel is similar to the formula given earlier for total annual vehicle miles of travel:

$$CVMT = PCT*AADT*LENGTH*260$$

where

$CVMT$ = total annual congested vehicle miles of travel,
PCT = percentage of daily traffic experiencing congested conditions, and
260 = days per year when recurring congested conditions occur.

Following the calculation of congested vehicle miles of travel, the next step in the analysis program is to calculate annual vehicle delay due to recurring congestion. To perform this calculation, some assumptions were required for vehicle operating characteristics under both congested and uncongested conditions. First, it was assumed that the average travel speed under uncongested conditions (levels of service A-C) is 55 mph. This is probably a conservative estimate. For V/C between 0.77 and 1.00, average travel speed was estimated from the curves shown in Figure 3-4 of the 1985 HCM (reproduced here as Figure 2) (*1*). As shown in Figure 2, for 70-mph design facilities, average travel speed varies between 30 and 54 mph for V/C between 1.00 and 0.77. Finally, for V/C greater than 1.0 (representing level-of-service F) an average travel speed of 20 mph was assumed. Selection of this value was largely subjective, but is in close agreement with other values given in the literature (*4*).

Total annual delay due to recurring congestion is estimated by the following equation:

$$DELAY = (IDEAL - ACTUAL)*PCT*AADT*260$$

where

$DELAY$ = annual delay due to recurring congestion,
$IDEAL$ = ideal section travel time per vehicle (average speed, 55 mph), and
$ACTUAL$ = actual section travel time per vehicle (less than ideal average speed).

This calculation is performed for each hour of congested travel on the freeway section.

Following the delay calculations, annual excess fuel consumption is calculated. A number of fuel consumption algorithms were studied to determine their applicability for use in this methodology. In particular, it was desired that an algorithm for the relationship between average speed and fuel consumption at congested freeway speeds be found, because average speed was already used in the analysis program for calculating delay. Unfortunately, current algorithms that describe this relationship apply only to speeds below 40 mph and are based on older vehicle fleets.

Therefore, a modified version of the fuel consumption data reported by Raus in 1981 (*5*) was used. In that study, fuel consumption values for average speeds between 1 and 35 mph were reported for the 1980 vehicle fleet. Data for average speeds between 20 and 35 mph were essentially linear. A linear regression best-fit analysis was applied to these data to determine an appropriate linear relationship that could be extended to average speeds up to 55 mph. The resulting expression is as follows:

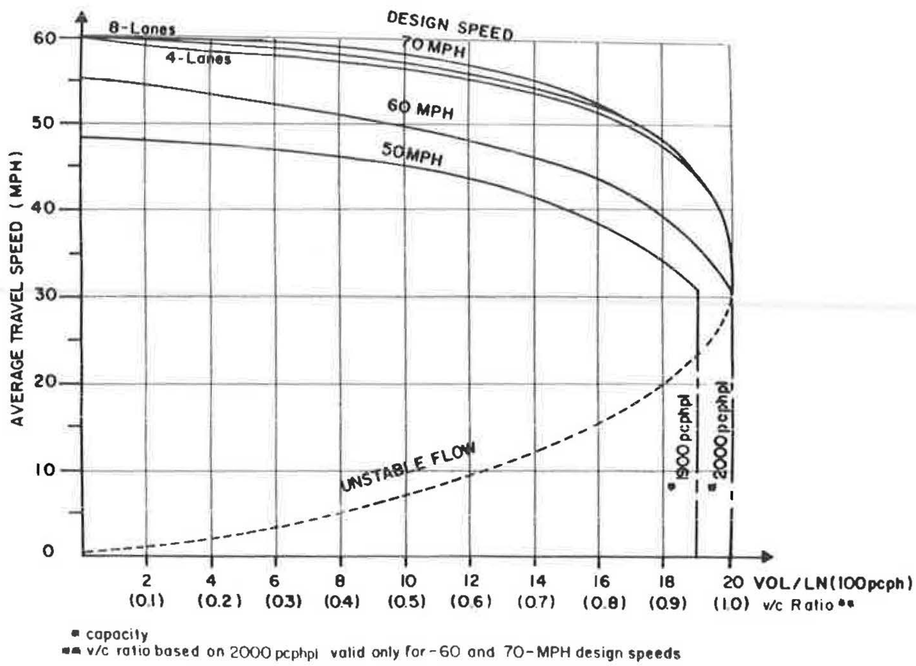

FIGURE 2 Speed-flow relationships under ideal conditions (*1*).

$$MPG = 8.8 + 0.25AVGSPD$$

where *MPG* is average fuel economy (in miles per gallon) and *AVGSPD* is average travel speed (in miles per hour).

Because this relationship is based on the 1980 passenger car vehicle fleet, fuel consumption estimates based on it may be slightly high. However, the presence of trucks in the traffic stream should tend to at least partially offset this potential error.

The next step in the analysis program is to estimate delay due to nonrecurring congestion caused by disabled vehicles and accidents. This portion of the methodology was based on previous work done on low-cost freeway incident management techniques (*6*). In this procedure, delay due to an incident can be estimated if information on freeway capacities and volumes and incident duration is known. The basic strategy was to apply the incident delay procedure repetitively for the freeway section for each hour of a typical day to estimate the total delay due to incidents.

This requires an average set of incident frequencies for various incident types, which was also available from the previous study on low-cost freeway incident management techniques. For the current methodology, two incident trees, one for freeways with adequate shoulders and one for freeways with no shoulders, were developed and are partially reproduced here as Figures 3 and 4. Each incident tree shows the breakdown by percentage of total incidents by incident type. Review of these figures indicates that a total of seven incident types were identified for freeways with adequate shoulders and five for freeways with no shoulders (by definition there can be no shoulder incidents on these facilities). The total incident rates associated with freeways with adequate shoulders and freeways with no shoulders are 200 and 79 incidents per million vehicle miles of travel, respectively. These incident frequencies were used directly in the analysis program.

As noted, freeway capacity under normal (nonincident) conditions either is an input data item or is calculated by the analysis program. Typical directional traffic volumes for each hour of the day are also derived as noted earlier. Freeway capacity during incident conditions, however, is not directly available and had to be derived. Figures for flow rates past one-lane incidents and shoulder accidents have been previously developed for typical four-, six-, and eight-lane freeways and are expressed in terms of vehicles per hour for typical capacity conditions (capacity = 1,850 vehicles/lane/hr) (*7*). For the analysis program, it was more useful to express these values in terms of percentage of total capacity remaining during an incident. It was also necessary to estimate values for shoulder disablements, two- and three-lane incidents, and freeway cross sections for up to 16 lanes. The final values used are shown in Table 1.

The average duration for incidents, including figures for both in-lane time and time spent on the shoulder, was estimated from several data sources on the basis of actual detection, response, and clearance times from operating urban freeways (*6-8*). These values are shown in Tables 2 and 3. Because the values shown in these tables are averages for each incident type, they are used in the analysis program each time an incident of that type occurs.

The overall operation of the incident delay portion of the analysis program includes (a) calculation of the number of occurrences per year for each incident type for each hour of the day using the incident trees shown in Figures 3 and 4, (b) calculation of the time until normal flow resumes following an incident by using freeway capacity and traffic volume information and the values in Tables 1-3, and (c) calculation of delay caused by the presence of an incident for each

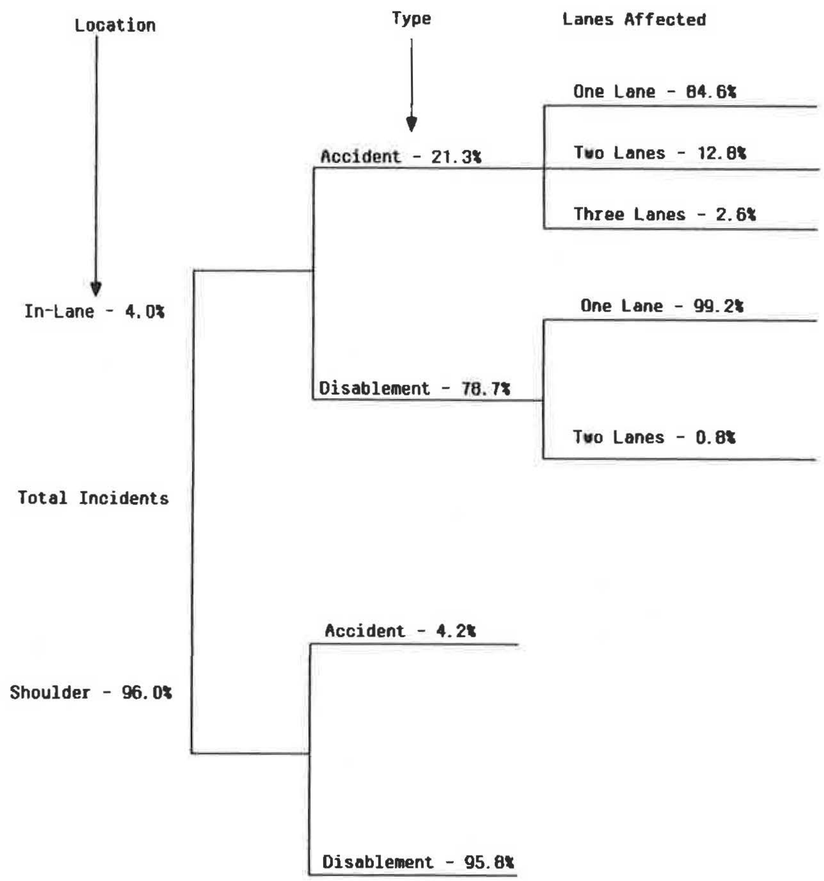

FIGURE 3 General incident tree (adequate shoulders).

incident type. Delay calculations are then expanded from a single incident occurrence to a full year by multiplying by the number of annual occurrences. A final step in the incident delay portion of the analysis program is to subtract from the incident delay total any recurring delay that would otherwise occur while the incident is present, to prevent double counting of the recurring delay.

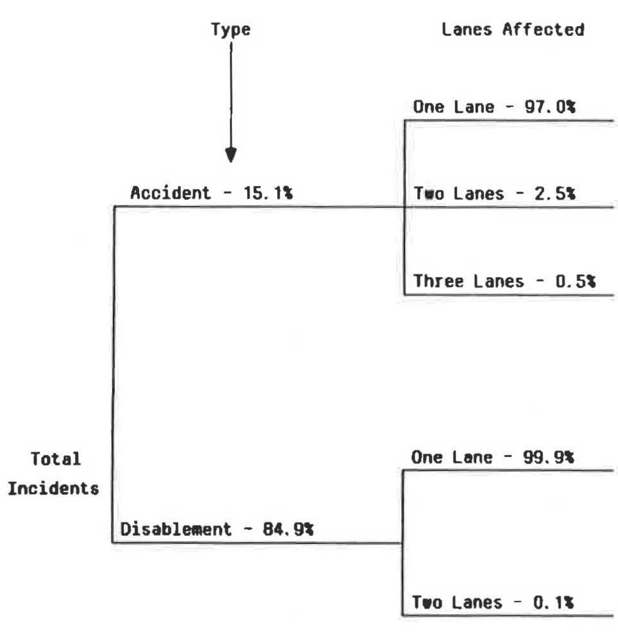

FIGURE 4 General incident tree (no shoulders).

Excess fuel consumption for nonrecurring congestion is calculated manually by assuming that the fuel consumption relationship previously expressed for recurring congestion also holds for nonrecurring congestion. Excess fuel consumption for nonrecurring congestion can thus be calculated for each of the freeway sections when delay due to incidents occurs.

The last step in the analysis program consists of calculating total user costs based on delay and excess fuel consumption. User costs due to time lost were calculated on the basis of a unit value of time derived by using the 1977 AASHTO Red Book (*9*). This publication quotes a 1977 value of time for 5 to 15 min of delay per trip (typical for an average urban freeway trip) as \$2.40/traveler hour for work trips. This value of time was expanded by using the Consumer Price Index to an October 1985 (*10*) value and an average vehicle occupancy of 1.25 was assumed, which yielded an average value of travel time of \$6.25/vehicle-hr. Other studies have calculated an even higher value of travel time (*11*). A value of \$1.00/gal was assumed for the cost of fuel.

TABLE 1 FRACTION OF FREEWAY SECTION CAPACITY AVAILABLE UNDER INCIDENT CONDITIONS

No. of Freeway Lanes in Each Direction	Shoulder Disablement	Shoulder Accident	Lane Blocked		
			One	Two	Three
2	0.95	0.81	0.35	0	0
3	0.99	0.83	0.49	0.17	0
4	0.99	0.85	0.58	0.25	0.13
5	0.99	0.87	0.65	0.40	0.20
6	0.99	0.89	0.71	0.50	0.25
7	0.99	0.91	0.75	0.57	0.36
8	0.99	0.93	0.78	0.63	0.41

TABLE 2 AVERAGE INCIDENT DURATION TIMES FOR FREEWAYS WITH ADEQUATE SHOULDERS

	Shoulder Disablement	Shoulder Accident	Disablement with Lane Blocked		Accident with Lane Blocked		
			One	Two	One	Two	Three
Detection	10	10	10	10	10	10	10
Response	10	10	10	10	10	10	10
Duration in lane after response	NA	NA	5	10	10	15	20
Total duration in lane	NA	NA	25	30	30	35	40
Duration on shoulder after response	10	20	15	15	20	25	30
Total	30	40	40	45	50	60	70

Note: All values are given in minutes. NA = not applicable.

TABLE 3 AVERAGE INCIDENT DURATION TIMES FOR FREEWAYS WITH NO SHOULDERS

	Disablement with Lane Blocked		Accident with Lane Blocked		
	One	Two	One	Two	Three
Detection	10	10	10	10	10
Response	10	10	10	10	10
Duration in lane after response	10	25	30	40	50
Total	30	45	50	60	70

Note: All values are given in minutes.

TABLE 4 URBAN FREEWAY CONGESTION STATISTICS

Item	1984	2005
Freeway miles	15,335	15,335
Vehicle miles of travel (millions)	276,645	410,987
Recurring congested vehicle miles of travel (millions)	31,486	98,280
Recurring delay (million vehicle hours)	485.0	2,048.6
Excess fuel consumption due to recurring delay (million gallons)	531.6	2,173.2
Delay due to incidents (million vehicle hours)	766.8	4,857.5
Excess fuel consumption due to incidents (million gallons)	845.9	5,143.9
Total delay (million vehicle hours)	1,251.8	6,906.1
Total excess fuel consumption (million gallons)	1,377.5	7,317.1
Total user costs ($ millions)	9,201.3	50,480.2

ANALYSIS RESULTS

As previously noted, the analysis program was applied to the HPMS database to obtain an estimate of national urban freeway congestion parameters. Table 4 shows the results of this analysis for both 1984 and 2005 (year 2005 AADT is an HPMS data item). These results are illustrative of the type of results one may expect when using the program for a specific freeway section or urban area. Those desiring further information on the results and conclusions of the study from which the methodology described is extracted should obtain a copy of the FHWA staff research study report *Quantification of Urban Freeway Congestion and Analysis of Remedial Measures* (*12*). The author may be contacted regarding the availability of this report.

SUMMARY AND FUTURE PLANS

This paper has described a computerized methodology developed for calculating the urban freeway congestion parameters of congested travel, recurring delay, nonrecurring delay, excess fuel consumption, and user costs. The methodology can be applied to single freeway sections or groups of freeway sections within an urban area. The methodology was used with a national database to quantify the urban freeway congestion problem.

The analysis program, as currently written, is tailored to the characteristics of the HPMS database. To permit its use by others, the program will be enhanced to allow direct user input. The user will be allowed to use default values for certain parameters, such as the value of travel time, or to substitute his own values. The revised program will run on an IBM-PC or compatible microcomputers and will be fully documented.

ACKNOWLEDGMENT

The author would like to express his appreciation to Denise Murray and Janet Crain for their assistance in preparing this paper for publication.

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The opinions and viewpoints expressed in this paper are those of the author and do not necessarily reflect the viewpoints, programs, or policies of the Federal Highway Administration.

Analysis of Freeway Reconstruction Alternatives Using Traffic Simulation

STEPHEN L. COHEN AND J. CLARK

Methods for evaluating traffic operations improvements for freeway reconstruction alternatives are discussed. It is asserted that traffic simulation provides a better approach to such analyses than the traditional *Highway Capacity Manual* (HCM). Several traffic simulation models are described. An application involving a congressionally mandated study of capacity improvements for a bridge in the Washington, D.C., area is described, for which the INTRAS freeway simulation model was chosen as the analysis tool. Required modifications to the INTRAS model and calibration and validation activities are described. In conclusion there is a description of the simulation experiment of the existing eastbound condition and five alternatives and the existing westbound condition and one alternative. The most interesting finding in this study was that an expansion of the eastbound span from three to five lanes performed no better than did several four-lane alternatives.

As the nation's Interstate freeway system ages, it is becoming necessary to rehabilitate sections of it that are wearing out, especially in dense urban areas. Although rehabilitation is usually considered to involve resurfacing, it is evident that it is cost-effective to pursue capacity improvements at the same time in order to relieve bottleneck locations and improve traffic operations.

In reconstruction projects involving extended sections of freeway, there are often a number of alternative approaches to improving traffic operations, and a procedure is needed to choose the best or most cost-effective, or both, among them.

ALTERNATIVE ANALYSIS METHODS

The method most used in the past to evaluate traffic operations improvements is given in the *Highway Capacity Manual* (HCM)(*1*). For freeways, in particular, it can be used to estimate the level of service (LOS) for a given bottleneck location both before and after a capacity improvement. However, as a stand-alone tool, it has a number of deficiencies:

1. Much of it is based on sparse data.
2. It is difficult to use it to gain insight into dynamic situations involving variable traffic demands because it is based on static situations.
3. It is difficult to use it to gain insight into the possible effects of one bottleneck location on another. This is especially true when such locations overlap, a condition that frequently occurs in dense urban areas with substandard geometrics.
4. The HCM assumes a constant correction factor for sluggish vehicles such as trucks and buses (for LOS > C) on level grades. This might be inadequate in many types of bottleneck locations.

A second method for alternatives analysis that has begun to generate some interest recently is traffic simulation. Simulation models have at least the potential for overcoming some or all of the deficiencies of the HCM noted. In the next section, some models that have been used for freeway analysis are described.

FREEWAY SIMULATION MODELS

Three models have been used for freeway traffic operations analysis.

FREQ

FREQ consists of a family of freeway simulation models, the latest of which are FREQ8PE and FREQ8PL (*2*). These models have been used to evaluate such measures as fixed-time ramp-metering plans, priority mainline lanes for high-occupancy vehicles (*3*), and priority ramp lanes for high-occupancy-vehicle ramp-meter bypass (*4*). The model is based on the principle that bottleneck sections produce shock waves when volume exceeds capacity.

The bottleneck capacities are obtained from the HCM. The FREQ models can be described as quasi-static macroscopic: quasi-static because changes in demand levels can only be input at specific times, macroscopic because the movement of individual vehicles is not modeled.

FREFLO

The FREFLO model (*5*) was developed from an earlier program, MACK (*6*). It has been used mostly to evaluate ramp-metering strategies. In particular, it has been used to evaluate real-time ramp-metering strategies (*7*) in which metering rates are adjusted in response to detector actuations from a surveillance system. The model is based on a conservation equation and a dynamic speed density equation.

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Nominal bottleneck capacities are input for all sections of the freeway, but the actual throughput obtained may differ from these values depending on conditions. The FREFLO model can be described as dynamic macroscopic: dynamic because changes in demand levels can be input at any time, macroscopic because the movement of individual vehicles is not modeled.

INTRAS

The INTRAS model (*8*) has been used to evaluate incident detection algorithms, real-time ramp-metering strategies, and the traffic operations implications of geometric reconstruction alternatives. The INTRAS model uses car-following and lane-changing laws to simulate the movement of individual vehicles. Thus, it can be described as dynamic microscopic: dynamic because changes in demand levels can be input at any time, microscopic because individual vehicle movements are modeled.

PROPOSED APPLICATION

As a demonstration of the use of traffic simulation to evaluate the effect on traffic operations of reconstruction alternatives involving capacity improvements, it was decided to use a simulation model to evaluate possible capacity improvements to two bridges in the Washington, D.C., area. A study of the feasibility of measures to improve the operational characteristics of both the Theodore Roosevelt (TR) Bridge on I-66 and the 14th Street Bridge on I-395 connecting the commonwealth of Virginia and the District of Columbia was mandated by the U.S. Congress in House Report 99-256. In this paper, the analysis of one of them, the TR Bridge, will be described in detail. Schematic diagrams of inbound and outbound I-66 and its approaches are shown in Figures 1 and 2.

The outstanding characteristics of the TR Bridge, particularly in the eastbound (a.m.-peak) direction, are substandard geometrics (such as closely spaced interchanges, short ramp acceleration lanes, short weaving sections), and

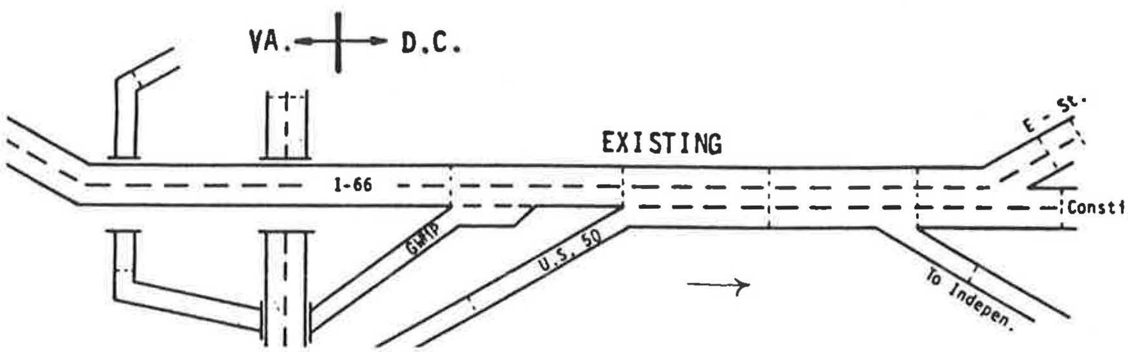

FIGURE 1 I-66 bridge inbound—existing condition.

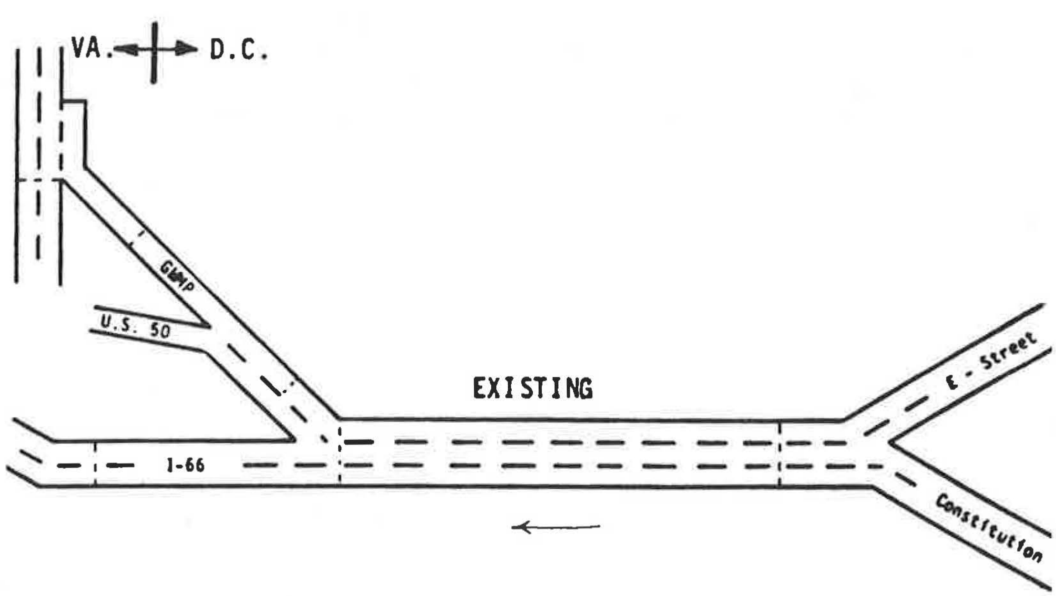

FIGURE 2 I-66 bridge outbound—existing condition.

heavy cross-weaving on the bridge structure itself. These characteristics illustrate the deficiencies in the use of the HCM and heavily influence the choice of simulation model to be used.

A preliminary analysis was done to determine the applicability of the three models described earlier. It was quickly determined that the two macroscopic models were inadequate because neither one merges or weaves, other than allowing the user to input capacities for such situations as given by the HCM. This is particularly deficient in cases where the HCM is weak, particularly in the cases of substandard merges and cross-weaves. The INTRAS model, on the other hand, has the capability of modeling such situations and was thus chosen for this study.

ADAPTATION OF INTRAS MODEL

Initially, a number of simulation runs made on the TR Bridge indicated that the INTRAS model was not properly representing the situation as observed in the field. A detailed investigation of the software was made, which involved a substantial number of computer runs with temporary print statements inserted. This investigation revealed that the INTRAS model had a number of deficiencies that had to be overcome if it was to be used successfully in this study. These deficiencies required either software modifications or special modeling of certain geometric conditions, the most important of which were as follows:

1. It was found that the lane-changing logic tended to give preference to the ramp over the mainline in ramp merge situations. An investigation of the logic showed that the maximum lane-change risk acceptable to a prospective lane changer was too large. This problem was solved by modifying the logic to make the maximum acceptable risk dependent on the length of the acceleration lane [a full discussion of the lane-changing logic in INTRAS may be found elsewhere (*8*, Vol. 1, pp. 139-144)].
2. The simulation program employs the trip distribution (TD) model used in FREQ8PL to assign vehicles entering on the mainline and entry ramps to exit ramps and the downstream mainline exit. This TD model is based on the user input volumes and freeway exit ramp fractions. A vehicle is assigned a destination when it enters the freeway on the basis of the origin-destination matrix elements of its origin point. When the vehicle passes an advanced-warning sign for its exit (this is input by the user at a location upstream from an off-ramp destination at which it is observed that vehicles begin to react to the off ramp), it is given an impetus to lane change to the left or right, depending on whether the exit is a left or right exit. It was observed that in tight cross-weaving situations such as are found on the TR Bridge, a number of vehicles in the model missed their exit. This problem was solved by increasing the maximum deceleration risk acceptable to the vehicle attempting to respond to the impetus in order to increase the rate of lane changing so that such vehicles obtain their proper lane more quickly.
3. At the time this study was done, the model was incapable of simulating interior lane additions and lane drops. These occur when, for example, the left lane of a two-lane right-hand entry ramp merges with the right lane on the freeway. This was handled by separating each lane of the ramp into a separate ramp (it should be noted that this problem was solved with a minor software modification).

This activity consumed a substantial amount of time and computer resources.

DATA ACQUISITION

One of the major problems encountered with performing a simulation analysis is the acquisition of data to run the model. The INTRAS model requires fairly detailed geometric and traffic information, such as

1. Location of such features as ramps and lane drops or additions,
2. Length of acceleration and deceleration lanes,
3. An estimate of how far upstream of an off ramp a vehicle destined for that off ramp begins to move over into the proper lane for exiting (position of advanced-warning sign),
4. Free-flow speeds,
5. Number of lanes,
6. Volumes on the on ramps and upstream end of freeway,
7. Fraction of mainline vehicle exiting at each off ramp, and
8. Percentage of trucks.

In addition, the user can override elements of the origin-destination (OD) matrix computed by the programs' trip distribution model. This capability was not used in the current study but is of special importance for cross-weaving situations in which there are data indicating that a model-computed OD matrix element is incorrect.

For this application, geometric and traffic data were obtained from a combination of aerial photographs, road maps, and field measurements. The traffic data were obtained from the District of Columbia Department of Transportation and the National Park Service and through the FHWA Direct Federal Programs Office and the Virginia Department of Highways and Transportation. These data consisted mostly of hand counts and volumes from road tubes.

CALIBRATION AND VALIDATION

One of the most important activities to be performed when a simulation model is applied is the calibration and validation of model outputs. This is done to ensure that the model is reflecting the real-world situation for the existing case. The major part of the calibration-validation activity in this study consisted of adjustment of model parameters, particularly the following-distance distribution (k) in the car-following law, in order to match the existing capacity as observed in the field with the capacity as predicted by the model. The calibration-validation adjustment for the bridge shown in Figure 1 was performed as follows:

1. The peak periods were determined from the traffic data and it was verified that the LOS during these periods was E or F. The eastbound direction had its peak period during the morning and the westbound direction had its peak period during the afternoon.
2. Simulation runs were made separately for the peak period for each bridge direction.
3. After each run, adjustments were made in the following-distance parameter k used in the car-following law until the model throughput agreed with the observed throughput.

ALTERNATIVES FOR THE TR BRIDGE

Eastbound Direction, Morning Peak (7:00-9:00 a.m.)

The existing condition can be seen in Figure 1. There is a total of five lanes entering the bridge, with three lanes on the bridge proper. Two of the lanes come from the I-66 mainline, one comes from a ramp off the George Washington (GW) Parkway, and two come from US-50 (although these narrow to one lane before actually entering the bridge). A total of five alternatives was examined and is shown in Figures 3-7.

- Alternative 1 consists of adding a fourth lane on the bridge from the Route 50 on ramp to the Independence Avenue exit. Thus, both lanes entering the bridge from Route 50 would be served.
- Alternative 2 consists of adding a fourth lane on the bridge from the GW Parkway entrance to the Independence Avenue exit. Thus, the existing GW Parkway-I-66 merge would be eliminated.
- Alternative 3 is Alternative 1 with the fourth lane extended to the Constitution Avenue exit.
- Alternative 4 is Alternative 2 with the fourth lane extended to the Constitution Avenue exit.
- Alternative 5 is Alternative 4 with two lanes on the entrance ramp from Route 50, the left lane of which merges with the right lane on the mainline (which came from the GW Parkway entrance).

An INTRAS run was made for each of the alternatives with the assumption that the OD matrix would remain fixed for all the alternatives. Although analysis of a possible traffic pattern change was outside the scope of work, it could well be that a reduction in weaving demand would produce substantial benefits in operational efficiency.

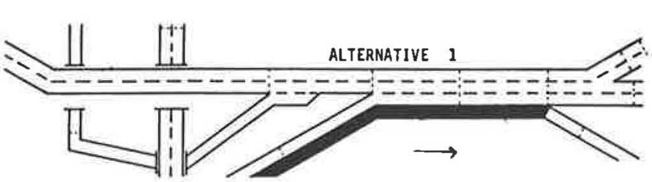

FIGURE 3 I-66 bridge inbound—Alternative 1.

Westbound Direction, Afternoon Peak (3:45-6:00 p.m.)

The existing condition can be seen in Figure 2. There is a total of four lanes entering the bridge, two from E Street and two from Constitution Avenue, with three lanes on the bridge itself. The left lane coming from E Street merges with the right lane coming from Constitution Avenue. One alternative, shown in Figure 8, was analyzed. This alternative consisted of adding a fourth lane to the bridge so that all four entry lanes are served. The right lane leads to the GW Parkway ramp as in the existing situation, and the second lane on the right leads only to Route 50 westbound and not to both Route 50 westbound and I-66 as in the existing condition.

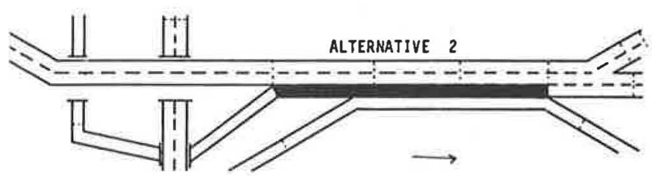

FIGURE 4 I-66 bridge inbound—Alternative 2.

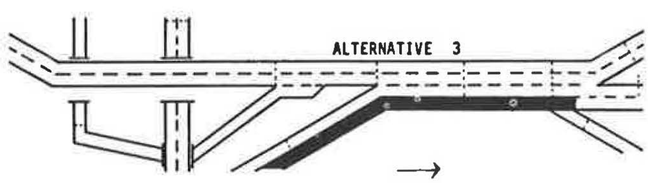

FIGURE 5 I-66 bridge inbound—Alternative 3.

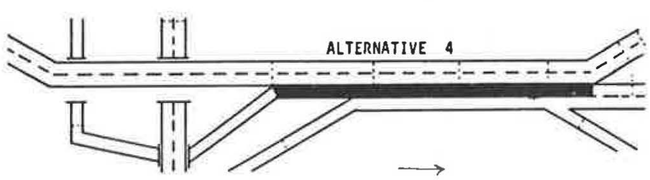

FIGURE 6 I-66 bridge inbound—Alternative 4.

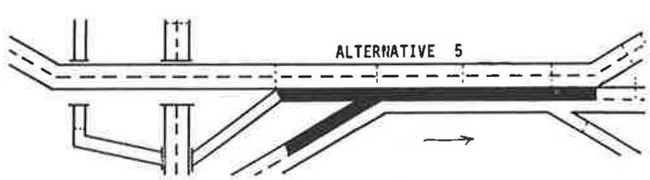

FIGURE 7 I-66 bridge inbound—Alternative 5.

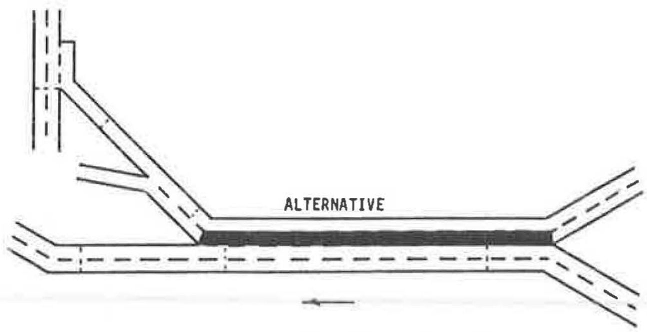

FIGURE 8 Alternative for I-66 bridge outbound.

RESULTS AND DISCUSSION

In order to compare the effectiveness of each alternative, the following measures of effectiveness (MOEs) were selected:

- Throughput on the bridge,
- Average speed on the bridge, and
- Queue lengths on the bridge approaches at the end of the simulation time period.

Other MOEs, such as total travel time, can be derived from average speed and volume.

Inbound Direction (7:00-9:00 a.m.)

Because it was found that the period from 7:00 to 8:00 a.m. was operating at a level of service better than E, only the results of the 1-hr period from 8:00 to 9:00 a.m., when the LOS was E or worse, are reported. Constant volumes were used for each 1-hr period because only hourly counts were available.

The results for the MOEs are shown in Table 1. From these results, the following observations can be made:

1. None of the alternatives were able to completely relieve congestion on both the bridge and all approaches.
2. Only Alternative 4 relieved congestion on the bridge. However, this was at the expense of maintaining a substantial queue on Route 50 because of geometric metering by the (existing) reduction from two lanes to one lane. However, the queue existing on the GW Parkway ramp was dissipated.
3. Alternative 1 fails to relieve bridge congestion because the extra lane only serves the 800 vehicles/hr destined for Independence Avenue, which is about the amount of extra vehicles that are able to get on from Route 50.
4. Alternative 2 fails to relieve bridge congestion for the same reasons as Alternative 1.
5. Alternative 3 fails to relieve bridge congestion because the extra lane causes more weaving and lets on more traffic from Route 50.
6. Alternative 5 fails to relieve bridge congestion because of more traffic from Route 50 and increased weaving.

After the conclusion that none of the foregoing alternatives completely relieves the situation, it was decided to try a five-lane alternative that would give each approach lane a separate lane onto the bridge. The left two lanes would go to E Street, the right three to Constitution Avenue. This was not considered among the original proposals because it involved substantial new construction (e.g., additional bridge piers). The result of this run was surprising. The capacity of the five lanes was no greater than that of Alternatives 3 and 4. However, analysis of the cross-weaving shows why the five-lane alternative fails. The OD demand is as follows:

	OD Demand (%) by Source		
Location	*Independence Avenue*	*Constitution Avenue*	*E Street Expressway*
I-66 mainline	9	30	61
GW Parkway ramp	9	31	60
Route 50 ramp	9	30	61

Analysis indicates the following:

1. Vehicles going from I-66 to Independence Avenue (202 vehicles) must change lanes at least three times instead of twice,
2. Vehicles going from the GW Parkway to Independence Avenue (149 vehicles) must change lanes twice instead of once, and
3. An additional 480 vehicles going from Route 50 to the E Street Expressway must make at least two lane changes.

All of these maneuvers must take place within a distance of 2,400 ft. Thus, any additional capacity that might be gained

TABLE 1 RESULTS FROM SIMULATION RUNS OF EXISTING CONDITION AND ALTERNATIVES 1-4 FOR INBOUND I-66 BRIDGE

Scenario	Demand from All Approaches (vehicles/hr)	Throughput on Bridge (vehicles/hr)	Avg. Speed on Bridge (mph)	Queue Length (vehicles) I-66	Route 50	GW Parkway Exit
Existing	6,555	4,879	15	599	800	105
Alternative 1	6,555	5,032	11	1,122	0	248
Alternative 2	6,555	5,040	11	319	800	0
Alternative 3	6,555	6,143	16	90	0	182
Alternative 4	6,555	5,761	33	0	800	0
Alternative 5	6,555	6,019	14	220	152	0

Note: Results are for peak hour from 8:00 to 9:00 a.m.

by adding a fifth lane is lost because of greatly increased weaving turbulence.

Outbound Direction

The results for the outbound direction are shown in Table 2. It can be seen that the extra lane relieved the congestion on the bridge and the queues on the approaches. It should be noted, however, that a potential problem could occur relative to the GW Parkway off ramp, which might back up because of congestion on the parkway, even though none was seen in the simulation run. This is because the demand on that ramp alternative will be very near the capacity of the parkway on ramp. The INTRAS merging logic gives only an approximate estimate of ramp capacity and, because this connector is short, even a relatively small error in capacity could generate a long-enough queue to block the right lane on the bridge.

CONCLUSIONS

From the results of this study, it has been shown that traffic simulation in general and the INTRAS model in particular provide a workable means of analyzing the traffic operations consequences for freeway reconstruction projects. For instance, the model was able to distinguish between alternatives that are rather similar (i.e., inbound Alternatives 1-4, which all involved adding one lane to the bridge). In addition, it was possible to show that a more costly alternative, namely, adding two lanes to the inbound bridge, was in fact no better from an operational standpoint than any of the others.

It can also be stated that a much more detailed operational performance analysis was possible than would have been available from a traditional analysis.

FUTURE DEVELOPMENTS

It should be pointed out that the INTRAS model is not yet fully operational. A considerable effort was required to adapt it to this application, both because of model deficiencies and because of inadequate explanations in the User's Manual. Thus, this effort should be regarded as an experiment that was successful because of a good understanding of the model's operation, which enabled the authors to get around its deficiencies. Thus, at this time, the model must be described as "user-unfriendly." FHWA currently has a project under way to upgrade the INTRAS model. It will be completely reprogrammed using modern structured design and all of the problems found in this and other studies will be remedied, making it user-friendly so that a detailed understanding of the model's operation will not be required in order to perform analyses such as that described in this paper. The new model will be called FRESIM and it is hoped that it will be available around January 1988.

TABLE 2 RESULTS FROM SIMULATION RUNS OF EXISTING CONDITION AND ALTERNATIVE FOR OUTBOUND I-66 BRIDGE

Scenario	Demand from All Approaches (vehicles/hr)	Throughput on Bridge (vehicles/hr)	Avg Speed on Bridge (mph)	Queue Length (vehicles)	
				E Street Expressway	Constitution Avenue
Existing	4,716	4,432	19	200	0
Alternative	4,716	4,643	28	0	0

Note: Results are for peak hour from 5:00 to 6:00 p.m.

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A Procedure for the Assessment of Traffic Impacts During Freeway Reconstruction

JOHN D. LEONARD AND WILFRED W. RECKER

Results are described of an effort directed toward developing a consistent methodology to assess the impacts of traffic disruption due to major transportation reconstruction and rehabilitation projects during the implementation period. In the approach taken, state-of-the-art traffic simulation models are used to estimate the performance of the transportation system during various construction phases. Alternative construction and traffic redirection strategies designed to minimize both the direct and indirect losses associated with the construction or rehabilitation are then evaluated with the development of a systematic, computerized procedure designed to (a) provide for the creation and comparison of multiple and "layered" reconstruction and rehabilitation scenarios, (b) minimize the required knowledge of both the detailed interactions with the model as well as with the host computer, and (c) produce meaningful, comparative outputs that assist in the selection of reasonable alternatives. The resulting modeling environment is viewed as a convenient tool to assist both the traffic engineer and the transportation planner in selection of reasonable reconstruction and rehabilitation plans and schedules.

There is general awareness that the infrastructure of many public works is in need of urgent repair and upgrading, if not complete renewal. Although standard techniques are available to forecast benefits and performance of the transportation network following completion of the projects, little is known about assessment of

1. The performance during the construction phase that, in the case of roadway reconstruction, may result in restrictions in capacity for periods longer than a year, and
2. The "malperformance cost" of the improvement in terms of the disruption due to construction.

In major urban areas, new highway construction and rehabilitation projects will have a profound impact on demands placed on the existing transportation network during progress of the projects.

This paper reports the development and implementation of a tool [CARHOP (Computer-Assisted Reconstruction—Highway Operations and Planning)] to assist the highway engineer or planner in the analysis of alternative highway reconstruction scenarios. An overview of this new tool is presented, followed by a more detailed description of the menu components of the CARHOP preprocessor, the interactive, menu-oriented component of the overall CARHOP environment. A sample interactive session with CARHOP is presented in a later section, followed by a presentation of an application of the CARHOP modeling environment to a sample network and a discussion of the results of two demonstration cases.

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OVERVIEW OF CARHOP ENVIRONMENT

The CARHOP environment provides a method for testing various transportation system management (TSM) alternatives related to the reconstruction of freeways and arterials in an existing transportation network. CARHOP combines the resources of several different computer simulation and optimization models into one interactive package, providing the user access to state-of-the-art, data-intensive computer simulation models in a manner that minimizes data preparation and input. In this way, one may focus more on the broader issues of reconstruction than on modification of large data sets and repetitive executions of the simulation. CARHOP allows the engineer to create reconstruction zones, modify their characteristics, and then evaluate the performance of the transportation network subject to the alteration of the surrounding arterial network characteristics and signal timings. Comparison statistics are compiled from each of the different submodels invoked on a subnetworkwide basis and along user-specified and computer-optimized detour routes around the reconstruction zone. The impacts of different driver behaviors and vehicle occupancies may also be tested within the modeling environment.

The CARHOP environment is separated into three independent computer packages:

- CARHOP preprocessor
- POSTCARS executor
- JOGGER postprocessor

Each of these packages is compatible with the others and is designed to be run in the order shown. Although it is not recommended, it is possible to execute any of the packages without the others.

The CARHOP preprocessor provides the user interface with the rest of the CARHOP environment. Designed as an interactive, menu-driven program, the preprocessor is responsible for managing all of the input data files and prompting the user for the various pieces of information, including the scope of the reconstruction, detour specifications (if any), alternative signal timings, and types of outputs to be provided. CARHOP is executed through a series of screen menus, making unnecessary the memorization of complex commands, and contains extensive error trapping to prevent erroneous data from being passed to subsequent modules. Although current data base information is accessible to the user (e.g., number of lanes, capacities, speeds) for use in designing reconstruction scenarios, modifications to the data base are simply logged during execution of the CARHOP preprocessor; the actual modifications are performed by subsequent modules, allowing time-intensive tasks to be performed in a noninteractive mode, which greatly speeds execution. Multiple scenarios may be tested at one time; in a matter of minutes, the user may design several alternative strategies for comparison.

The POSTCARS executor is responsible for taking the scenarios described by the preprocessor and performing the different operations requested. In the process, several data sets may be created, several different simulations may be performed, and extensive outputs may be generated. POSTCARS coordinates the execution of these simulations and performs the necessary data set conversions. At each stage of POSTCARS execution, information on all operations performed is stored in a log. This provides a hard copy of the scenario session, together with any special messages or conditions generated by subprograms. POSTCARS is designed to operate in a batch environment, without user intervention.

JOGGER, the CARHOP postprocessor, compiles statistics and generates comparative outputs based on the statistics generated in the POSTCARS executor. Statistics are presented on a link-by-link basis as well as on a subnetwork-wide basis. If detour outputs are requested, statistics are compiled along each detour route and compared with those of the original routes. Descriptions of each scenario are generated from the preprocessor outputs, providing the user with a hard copy of the actual scenario descriptions processed by the POSTCARS executor. As with the POSTCARS executor, JOGGER is designed to operate in the batch computer environment, requiring no user interaction.

Rather than being an explicit simulation model, the CARHOP environment is an organizer and executor. The TRAF modeling system (*1*) is used as the base simulation model for the CARHOP environment. Used like a chalkboard, TRAF is run on the base-case scenario; changes simulating network modifications associated with the reconstruction scenarios are then made to the base case. In addition, several support packages are included to facilitate data transfer among these simulation models as well as to create new data sets based on the changes specified by the user. From the options requested in a scenario log file created by the CARHOP preprocessor and the data requirements of each of the submodels, some or all of these programs may be executed. The TRANSYT-7F simulation model (*2*) is included in the CARHOP environment to provide optimized traffic signal timings along a user-specified detour route. This and other simulation models employed are used to generate modified TRAF data sets, reflecting changes in signal timings, detour routes, and network coding. Although the TRAF modeling system comprises many different simulation models of varying degrees of statistical resolution, CARHOP supports only three (all in TRAFLO of TRAF): Level 2 (arterial package), FREEFLO (freeway package), and TRAFFIC (traffic assignment). FREEFLO is used to gather statistics along the freeway subnetwork. This model is based on a macroscopic representation of traffic flow. For the arterial portion of the transportation system, the TRAFLO Level-2 model is employed. This model is most similar to TRANSYT-7F and, although also macroscopic, provides a relatively comprehensive set of vehicle and person travel statistics. In addition to the simulation models of TRAF, traffic assignment for the CARHOP environment is provided by the TRAF implementation of TRAFFIC (*3*), which employs an equilibrium-based assignment algorithm.

COMPONENTS OF THE CARHOP PREPROCESSOR

The CARHOP preprocessor organizes options within CARHOP into 10 distinct areas of concentration:

1. Data-base selection,
2. Freeway-incident specification,
3. Reconstruction-zone specification,
4. Detour-route specification,
5. Signal-timing alteration,
6. Physical-network alteration,
7. User-attribute alteration,
8. Transit-system alteration,
9. Graphics specifications, and
10. Scenario processing.

These options provide a wide range of data-set manipulation features. Any or all of these options may be used in the specification of a particular TSM strategy scenario. A brief description of each option follows. Associated with each description is the visual display of the preprocessor to the user.

Data-Base Selection

Data-base selection performs the role of "bookkeeper" in the processing of multiple scenarios and is executed before the creation of any CARHOP scenario:

1. Select Base Scenario

Current Scenario: NONE Default Data Base: NONE

- Change Default Data Base
- Create New Scenario
- RETURN TO MAIN MENU

Freeway-Incident Specification

Freeway-incident specification allows the user to create an incident on the freeway network. Examples of incidents include stalled vehicles, collisions, bottlenecks, and so on. The user need only specify the link on which the incident will occur together with the new estimated capacity.

2. Create Freeway Incident

Current Scenario: DT003 Default Data Base: SYMNET.DAT

- Create Freeway Incident
- RETURN TO MAIN MENU

Reconstruction-Zone Specification

Reconstruction-zone specification creates the actual reconstruction zone in the data base. It prompts the user for the location of the reconstruction zone and the system alterations resulting from the type of activity that is planned. These alterations include decreasing the lane capacities, lane and ramp closures, and estimated speed reduction zones.

3. Create Reconstruction Zone

Current Scenario: DT003 Default Data Base: SYMNET.DAT

- Arterial Sub-Network Reconstruction
- Freeway Sub-Network Reconstruction
- RETURN TO MAIN MENU

Detour-Route Specification

Detour-route specification allows the user to test various detour strategies associated with the reconstruction zone created in the previous module. There are several levels to this module. First, the user has the option of entering no detours at all. Statistics compiled at this level will provide a measure of the unmitigated impact of the reconstruction. The second level of this module provides the option of a user-specified detour route. Single or multiple routes may be entered. In addition, the user may compare the effectiveness of several different detours (multiple runs). The third level of this module allows the creation of detours based on the reallocation of trips in the traffic assignment model. This option, used in conjunction with other CARHOP options, allows the testing of the effects of additional lanes, new signal timings, no-truck restrictions, modified lane stripings, and other innovative strategies for improving traffic flow on the surrounding surface street system.

4. Detour Specifications

Current Scenario: DT003 Default Data Base: SYMNET.DAT

- Short-Term User-Specified Detours
- Short-Term Optimized Detours
- Long-Term User-Specified Detours
- Long-Term Optimized Detours
- RETURN TO MAIN MENU

Signal-Timing Alteration

Signal-timing alteration allows the testing of the impact of signal timings on the performance of the reconstruction strategy. Signal timings may be left as is or may be optimized. The effects of cycle length and signal progression may also be explored with this module. These modifications may be made to the existing network, to the network containing the reconstruction zone alone, or to the specified detour route. Signal progression and optimum cycle length calculations may be performed on individual intersections, the network immediately surrounding the reconstruction zone, or along the specified detour route.

5. Alter Signal Timings

Current Scenario: DT003 Default Data Base: SYMNET.DAT

- Alter Individual Intersection Timing
- Individual Intersection Optimization
- Global Intersection Optimization
- Arterial Corridor Optimization
- RETURN TO MAIN MENU

Physical-Network Alteration

Physical-network alteration provides a method of testing various supply-side TSM strategies. Two-way streets may be converted to one-way streets, and vice versa. The effects of turning restrictions, intersection channelization, parking restrictions, number of lanes, and lane widths may be explored. This module may also be implemented with other modules in CARHOP to estimate the impacts of complex reconstruction scenarios.

6. Alter Physical Network

Current Scenario: DT003 Default Data Base: SYMNET.DAT

- Alter Freeway Characteristics
- Alter Arterial Characteristics
- RETURN TO MAIN MENU

User-Attribute Alteration

User-attribute alteration provides for the investigation of the effects of different fleet compositions (car-to-truck ratios, etc.) and .vehicle types on system performance. Different driver behaviors (start-delay at intersections, etc.) may also be input.

7. Alter User Attributes

Current Scenario: DT003 | Default Data Base: SYMNET.DAT

- Alter User Attributes
- RETURN TO MAIN MENU

Transit-System Alteration

Transit-system alteration allows for modification to be made to the transit data base. Bus routes may be added and deleted. Average bus occupancies may be modified to test the effects of improved bus ridership on system performance.

8. Alter Transit System

Current Scenario: DT003 | Default Data Base: SYMNET.DAT

- Alter Transit System
- RETURN TO MAIN MENU

Graphics Specifications

The graphics module of CARHOP converts the physical network characteristics into computer-plotter instructions for later processing. Detour routes, bus routes, and changes in travel patterns between origins and destinations may be represented.

9. Produce Network Graphics

Current Scenario: DT003 | Default Data Base: SYMNET.DAT

- Create Graphics Instructions
- RETURN TO MAIN MENU

Scenario Processing

Once the single or multiple reconstruction zones, detour routes, and other options have been executed (as desired), the scenario-processing option creates a "logfile" containing all the information in a form to be read by the POSTCARS executor. (No changes to the base case are made during execution of the CARHOP preprocessor.) A series of sub-options are available giving the user the ability to create more scenarios, delete the scenario just created, or exit CARHOP.

10. Produce Job Control Instructions (EXIT)

Current Scenario: DT003 | Default Data Base: SYMNET.DAT

- Process Current Scenario and RETURN
- Process Current Scenario and EXIT
- DELETE Current Scenario (TOSS OUT)
- RETURN To Main Menu
- QUIT CARHOP (ABORT)

SAMPLE SESSION

In this section a sample session in CARHOP is provided in which a reconstruction scenario is created. The scenario created will consist of a single reconstruction zone with one user-specified detour provided as a means of routing a portion of the highway traffic around the reconstruction area.

CARHOP specification of the reconstruction zone and corresponding detour route consists of the following steps, performed in sequential order. Descriptions of each, together with some accompanying visual terminal displays, follow.

1. Specify the default data base and select menu options and scenario data base,
2. Define reconstruction zone and specify zone characteristics,
3. Specify detour route, and
4. Process current scenario and exit.

Specify the Default Data Base and Select Menu Options and Scenario Data Base

The session begins with a display that announces the actuation of the CARHOP environment (Figure 1). A prompt is then given for the user to provide a name for the default data base containing the TRAF data base information. The CARHOP preprocessor requires a default data base as input. This default data base consists of a standard TRAF input data set containing the network geometrics, signal timings, and origin-destination (O-D) information of the study area. For the demonstration of the CARHOP environment, a hypothetical network was devised. It was chosen to highlight various features of the simulation environment and to demonstrate the behavior of the simulations under varying reconstruction conditions. After receiving the name of the default data base, CARHOP will ask for any other scenario files to be used during the session.

With the names of the default data base or bases entered, the screen is cleared and the main CARHOP preprocessor menu is displayed. This menu displays the 10 options available as well as a status line showing the current scenario and default data base. Menu options are selected by moving the pointer until it is aligned with the option to be performed.

The first task in creating a reconstruction scenario file is to specify from which base the scenario file is to be created. This is accomplished by executing Menu Option 1. The Option 1 submenu provides the user with three choices: Change Default Data Base, Create New Scenario, and Return to Main Menu.

Upon selection of the Change Default Data Base option, CARHOP will display the choices available. This list will include all the file names entered by the user when the program was started.

After the data base has been selected, an information page will be displayed on the screen (Figure 2) showing important data-base information such as run name, user name, agency, run date, and run type. This information is provided to assist the user in the proper selection of the default data base. The next step is to define a working-scenario file. If a default scenario was just selected, CARHOP automatically prompts the user for the name of the working scenario. This option may also be performed without selecting a default data base by marking the appropriate option. Upon selection of a working-scenario file, the user enters the appropriate file information.

Define Reconstruction Zone and Specify Zone Characteristics

To specify a reconstruction zone, the user must select Option 3 of the main menu. CARHOP will then display the associated submenu, which allows three further choices: Arterial Subnetwork Reconstruction, Freeway Subnetwork Reconstruction, and Return to Main Menu (see Reconstruction-Zone Specification in previous section). CARHOP next prompts the user for the upstream and downstream mainline nodes encompassing the reconstruction area. If a reconstruction zone is to encompass several mainline links, each link is entered separately. After the downstream node number has been typed, CARHOP scans the default data

```
                     WELCOME TO

 CCC      A      RRRR    H   H    OOO    PPPP
C   C    A A     R   R   H   H   O   O   P   P
C       A   A    R   R   H   H   O   O   P   P
C       AAAAA    RRRR    HHHHH   O   O   PPPP
C   C  A     A   R   R   H   H   O   O   P
 CCC   A     A   R    R  H   H    OOO    P



           Computer
             Assisted
               Reconstruction Strategies for
                 Highway
                   Operations and
                     Planning
           Institute of Transportation Studies
             University of California, Irvine
```

FIGURE 1 Display announcing actuation of CARHOP environment.

```
              1. Select Base Scenario

Current Scenario: NONE          Default Data Base: NONE

            Data Bases To Choose From:

     ===>> X   SYMNET.DAT


              PROCESSING RECORD...    9

              SELECT AS BASE? ("Y" if yes)   Y

Run Name -->   TEST NETWORK: SYMMETRICAL WITH ONE FREEWAY, PASSER

          User Name -->  JOHN D. LEONARD
          Agency    -->  UC IRVINE
          Run Date  -->  10  05  85

          Run Type  -->  ASSIGNMENT AND SIMULATION
```

FIGURE 2 Information page display, Option 1.

base for the link. If it is not found, an appropriate message is displayed and the user is asked to try again.

If the link is found, an information page will be displayed showing the current characteristics of the input link (Figure 3). These characteristics are determined from information contained in the default data base and include the number of regular-use lanes, the number of special-purpose lanes, the nominal lane capacity in vehicles per hour, and the free-flow speed in miles per hour. CARHOP now prompts:

CREATE RECONSTRUCTION ZONE? (Y/N)

Given a reply of 'Y,' CARHOP will create a reconstruction zone. CARHOP will then prompt the user for the number of lanes closed, the new lane capacity, and the new free-flow speed.

Once this has been completed, CARHOP returns the user to the reconstruction submenu. Any reconstruction zones that may have been specified are listed as a reminder of the work already completed. The user may specify as many reconstruction zones as desired in any one scenario. When all of the reconstruction zones have been specified, the option Return to Main Menu is executed.

Specify Detour Route

With a reconstruction zone specified, corresponding detours may also be specified. This is achieved by entering Option 4 from the main menu: Detour Specifications. If this is chosen, the detour specification submenu is displayed (see Detour-Route Specification in previous section). There are two suboptions currently implemented: Short-Term User-Specified Detours and Long-Term Optimized Detours. In this example the first suboption will be described.

CARHOP now prompts the user for the original route, which is entered node by node. It must begin and end on the freeway subnetwork. Next CARHOP will prompt the user for the detour route (Figure 4), which must begin and end with the same nodes as the original route. The detour route may leave the freeway and go to the arterial subnetwork but must reenter the freeway and end at the same node as the

```
                 3. Create Reconstruction Zone

Current Scenario: DT003          Default Data Base:    SYMNET.DAT

                 o Freeway  Sub-Network Reconstruction


     CURRENT CHARACTERISTICS FOR LINK (  507 ,   508 )

          Regular Use Lanes    ..... 3
          Special Purpose Lanes ... 0

 Nominal Lane Capacity (veh/hr)....2000
          Free-Flow Speed (MPH)....  55
```

FIGURE 3 Current characteristics of input link.

```
                 4. Detour Specifications

Current Scenario: DT003          Default Data Base:    SYMNET.DAT

                  o Short-Term User-Specified Detours


                    Enter DETOUR Route.  It must begin
                    and end with the SAME nodes as the
                    ORIGINAL  route  just   specified.

     Starting At ===>   527           To ===>   45
              To ===>   506           To ===>   7511
              To ===>   7506          To ===>   511
              To ===>   43            To ===>
              To ===>   46
              To ===>   47
              To ===>   48
              To ===>   49
              To ===>   50

          Percentage of vehicles using route ==> 10
```

FIGURE 4 Specification of detour route.

original route. When both routes have been entered, each route is checked for continuity, that is, to determine that each node pair forms a link that exists in the default data base and is connected to the preceding link. If the routes pass this test, the user is prompted for the percentage of vehicles using the detour route. This percentage is used to calculate the volume of trips to be routed from the original route to the detour route.

Process Current Scenario and Exit

The final operation to be performed in this example scenario is to process the information that has just been entered. The processing combines the separate instructions from each different option into a single scenario logfile that can later be used by the POSTCARS executor and JOGGER postprocessor.

Selecting Option 10 displays the Process Current Scenario submenu. There are five choices available from this submenu. To create more scenarios, the Process Current Scenario and RETURN option would be selected. Had any errors or undesirable selections been made, Option 3, DELETE Current Scenario (TOSS OUT), would be selected. If Option 10 had inadvertently been chosen, the user would return to the main menu to select other options. The QUIT CARHOP option is provided as a means to stop execution of the current CARHOP session without saving the current scenario. The option Process Current Scenario and EXIT produces comparisons of the reconstruction scenario to the base case when the POSTCARS executor and JOGGER postprocessor are executed.

DEMONSTRATION OF CARHOP RECONSTRUCTION METHODOLOGY

To demonstrate use of the CARHOP environment in the study of freeway reconstruction and its impact on the surrounding arterial subnetwork, two case studies are presented:

1. A range of freeway reconstruction scenarios with implementation of the long-term detour specification of CARHOP and
2. Fixed freeway reconstruction (two open mainline lanes) with a ranged percentage of traffic along the detour route.

The demonstration network shown in Figure 5 includes a typical freeway section overlaid on a grid-pattern arterial network. The network was designed to be symmetrical about the major and minor axes to simplify interpretation of the operation of the underlying TRAF simulation modeling system. The freeway subnetwork consists of a single section of freeway with three through lanes in each direction bisecting an arterial grid subnetwork. The arterial subnetwork comprises four east-west corridors and five north-south corridors and connects to the freeway subnetwork at three alternating interchanges, each of symmetrical diamond shape. Streets on the arterial subnetwork are spaced at half-mile intervals. In terms of the TRAF data base designations, the arterial portion of the study area is coded as a Level-2 subnetwork and consists of 31 links, 26 regular nodes, 18 entry-exit nodes, and 12 interface nodes. The freeway portion is coded as a FREEFLO subnetwork and consists of 30 links, 16 regular nodes, 2 entry-exit nodes, and 12 interface nodes.

Before the test cases were run, the signal timings were optimized by using the PASSER II-80 simulation model along all north-south corridors of travel. Each of the test cases was chosen to demonstrate a particular facet of the CARHOP environment and the performance of the TRAF simulation modeling system under a range of inputs.

Test Case 1

In Test Case 1 CARHOP is used to assess the effects of freeway reconstruction with the long-term detour route option specified. In each of the scenarios of Test Case 1, the freeway mainline traffic flow is constrained and the traffic is allowed to reroute around the bottleneck.

Test Case 1 consists of five separate CARHOP scenarios. A reconstruction zone of three successive links is established along both directions of the freeway. The number and severity of constraints in each scenario are sequentially increased from minor speed and capacity [in vehicles per hour per lane (vphpl)] constraints to full closure of the freeway mainline (Table 1). The link constraints shown in Table 1 apply to all links in the reconstruction zone; six freeway links are directly affected by the constraints. Estimated link

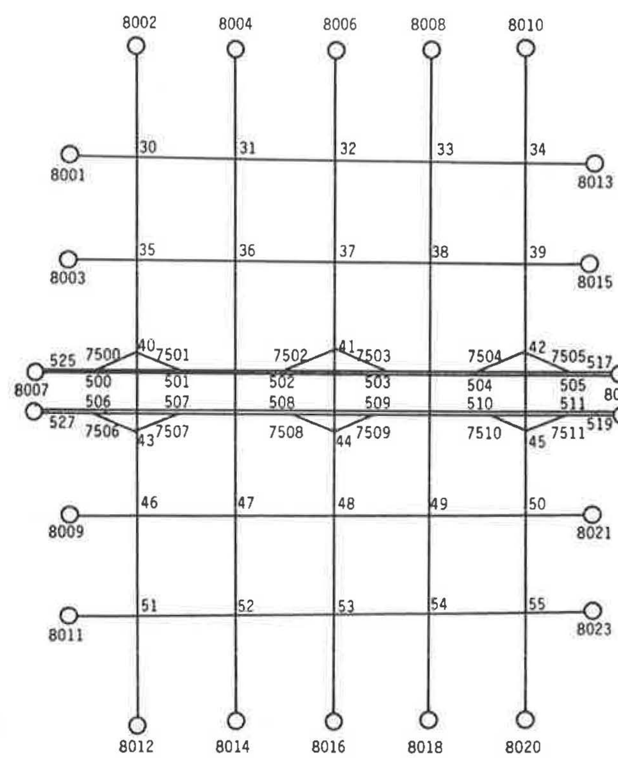

FIGURE 5 Demonstration network.

TABLE 1 SCENARIOS FOR TEST CASE 1

Scenario No.	Scenario Name	Link Capacity (vphpl)	Free-Flow Speed (mph)	Open Lanes
Base	SYMNET	2,000	55	3
1	MT001	1,000	45	3
2	MTOO2	1,400	40	3
3	MT003	1,400	40	2
4	MT004	1,400	40	1
5	MT005	1,400	40	0

capacities are taken from the *Highway Capacity Manual* work-zone estimates (*4*).

Figure 6 shows a sample of the scenario summary output for Scenario MT003 of Test Case 1. The tables in Figure 8 compare the characteristics of the link in the scenario with those in the base case.

For example, Link 507, 508 has two lanes in Scenario MT003, whereas in the base case it has three. The free-flow capacity of the link was reduced from 2,000 vphpl to 1,400 vphpl. The results of the simulation indicate that the reconstruction link in the base case has a simulated speed of 33.8 mph; during reconstruction the simulation produces a speed of 5.4 mph. Other statistics shown in the reconstruction zone summary include the number of trips through the zone, the volume/capacity ratio, and minutes per trip required to pass through the zone.

The final section of the scenario summary report contains the detour summary, which includes the type of detour option selected, the number of detour routes, and a list of the original and suggested detour routes for all O-D pairs affected by the construction.

Figure 7 gives a tabular comparison of four cumulative subnetworkwide statistics, as well as global totals for the entire transportation system. Histogram comparisons for each performance measure by subnetwork type and global network are also provided by CARHOP (Figures 8 and 9).

Test Case 2

In the second test case CARHOP is applied to evaluate the effectiveness of a particular short-term user-specified detour during freeway reconstruction. Test Case 2 consists of five CARHOP scenarios. Reconstruction zones comprising three successive links are established along the south section of freeway. Test Case 2 examines a fixed-reconstruction-zone strategy: two lanes are open along the freeway with a free-flow speed of 40 mph and a capacity of 1,400 vphpl; the percentage of freeway trips routed from the freeway to the arterials is gradually increased. The scenarios for Test Case 2 are shown in Table 2.

Figure 10 shows that the summary output freeway performance improves as the percentage of trips routed along the detour increases. The associated freeway speeds increase from 11.5 mph with no trips detoured to 13.1 mph with 20 percent detoured. The number of vehicle hours also decreases, from 2,200.6 with no trips detoured to 1,867.0 with 20 percent detoured.

When no traffic is routed from the freeway to the arterials in the short term, the speeds along the arterial are relatively unaffected. As the percentage of trips routed along the detour is increased, the arterial subnetwork average speed begins to decrease, ranging from 14.4 mph with 5 percent detours to 10.1 mph with 20 percent detours. The number of vehicle trips in the arterial subnetwork decreases slightly.

The individual scenario reports, one of which is shown in Figure 11, contain a table of the travel time along routes, comparing the time, in minutes per trip, required for a vehicle to travel along the original and detour routes. This statistic demonstrates the trade-off occurring during the routing of freeway trips along the detour route. For the base case, a vehicle requires 4.8 min to traverse the original route through the reconstruction zone. When the reconstruction constraints are imposed and no traffic is allowed to detour, the time required increases to 31.8 min. By comparison, travel time along the detour route is 9.8 min in the base case. As the percentage of trips routed along the detour route is increased, conditions along that route become congested, resulting in an increase in travel time along the route. With 5 percent of the freeway traffic routed along the detour, the travel time increases to 27.7 min per trip, whereas the travel time along the original route decreases to 29.9 min. With 10 percent of the freeway trips routed along the detour, the fastest route again becomes the freeway, with a trip taking 29.8 min along the original route and 67.1 min along the detour route. The travel times when 15 and 20 percent of the freeway trips are routed along the detour are 125.9 and 202.5 min, respectively. The equilibrium that can be expected to be realized under these conditions thus is approximately 5 percent of the freeway traffic selecting the detour route.

TABLE 2 SCENARIOS FOR TEST CASE 2

Scenario No.	Scenario Name	Link Capacity (vphpl)	Free-Flow Speed (mph)	Open Lanes	Percentage of Trips Detoured
Base	SYMNET	2,000	55	3	N/A
1	DT001	1,400	40	2	0
2	DT002	1,400	40	2	5
3	Dt003	1,400	40	2	10
4	DT004	1,400	40	2	15
5	DT005	1,400	40	2	20

INFORMATION SUMMARY FOR SCENARIO:MT003

SCENARIO ----> MT003
DESCRIPTION ----> RECON BOTH DIRECTIONS:
LANES REDUCED TO 2
DATA CREATED ----> 10 05 85
USER NAME ----> JOHN D. LEONARD
AGENCY ----> UC IRVINE -- ITS

CREATED FROM BASE NETWORK: SYMNET

LENGTH OF SIMULATION (IN SECONDS) = 3600

RECONSTRUCTION ZONE SPECIFIED AT LINK (507, 508)

		SYMNET	MT003
NUMBER OF LANES	=	3	2
FREE FLOW CAPACITY	=	2000	1400
SPEED THROUGH ZONE	=	33.8	5.4
TRIPS THROUGH ZONE	=	4496.0	2799.0
VOLUME/CAPACITY	=	0.7493	0.9996
MINUTES PER TRIP	=	0.8886	5.5256

RECONSTRUCTION ZONE SPECIFIED AT LINK (508, 509)

		SYMNET	MT003
NUMBER OF LANES	=	3	2
FREE FLOW CAPACITY	=	2000	1400
SPEED THROUGH ZONE	=	33.7	13.6
TRIPS THROUGH ZONE	=	4270.0	2398.0
VOLUME/CAPACITY	=	0.7117	0.8564
MINUTES PER TRIP	=	0.8906	2.2060

RECONSTRUCTION ZONE SPECIFIED AT LINK (509, 510)

		SYMNET	MT003
NUMBER OF LANES	=	3	2
FREE FLOW CAPACITY	=	2000	1400
SPEED THROUGH ZONE	=	34.6	19.8
TRIPS THROUGH ZONE	=	4463.0	2619.0
VOLUME/CAPACITY	=	0.7438	0.9354
MINUTES PER TRIP	=	0.8672	1.5136

RECONSTRUCTION ZONE SPECIFIED AT LINK (504, 503)

		SYMNET	MT003
NUMBER OF LANES	=	3	2
FREE FLOW CAPACITY	=	2000	1400
SPEED THROUGH ZONE	=	33.8	5.4
TRIPS THROUGH ZONE	=	4491.0	2799.0
VOLUME/CAPACITY	=	0.7485	0.9996
MINUTES PER TRIP	=	0.8874	5.5256

RECONSTRUCTION ZONE SPECIFIED AT LINK (503, 502)

		SYMNET	MT003
NUMBER OF LANES	=	3	2
FREE FLOW CAPACITY	=	2000	1400
SPEED THROUGH ZONE	=	33.7	13.6
TRIPS THROUGH ZONE	=	4265.0	2398.0
VOLUME/CAPACITY	=	0.7108	0.8564
MINUTES PER TRIP	=	0.8905	2.2060

RECONSTRUCTION ZONE SPECIFIED AT LINK (502, 501)

		SYMNET	MT003
NUMBER OF LANES	=	3	2
FREE FLOW CAPACITY	=	2000	1400
SPEED THROUGH ZONE	=	34.6	19.8
TRIPS THROUGH ZONE	=	4458.0	2618.0
VOLUME/CAPACITY	=	0.7430	0.9350
MINUTES PER TRIP	=	0.8671	1.5118

DETOUR OPTION SELECTED ----> LONG-TERM CARHOP-OPTIMIZED

NUMBER OF DETOUR ROUTES ----> 64

ORIGINAL ROUTE:
8001 30 35 40 43 7507 507 508 509 510 511 519 8019

DETOUR ROUTE:
8001 30 31 36 37 38 39 42 45 7511 511 519 8019

ORIGINAL ROUTE:
8001 30 35 40 43 7507 507 508 509 510 7510 45 50 55 8020

DETOUR ROUTE:
8001 30 31 36 37 38 39 42 45 50 55 8020

ORIGINAL ROUTE:
8001 30 35 40 43 7507 507 508 509 510 7510 45 50 8021

DETOUR ROUTE:
8001 30 31 36 37 38 49 50 8021

ORIGINAL ROUTE:
8001 30 35 40 43 7507 507 508 509 510 7510 45 50 55 8023

DETOUR ROUTE:
8001 30 31 36 37 38 49 54 55 8023

FIGURE 6 Scenario summary output.

	SYMNET	MT001	MT002	MT003	MT004	MT005
ARTERIAL SUBNETWORK						
AVERAGE SPEED	15.9	13.5	10.8	9.2	3.5	1.8
VEHICLE TRIPS	19747.	18135.	17527.	17107.	14293.	10664.
VEHICLE HOURS	2042.9	2472.8	2947.6	3425.6	6723.3	9392.1
VEHICLE MILES	32531.7	33472.6	31954.7	31516.5	23391.0	16845.4
FREEWAY SUBNETWORK						
AVERAGE SPEED	34.3	33.4	31.1	10.2	2.7	0.8
VEHICLE TRIPS	7689.	7637.	7573.	6326.	3549.	1710.
VEHICLE HOURS	876.6	835.8	860.2	2149.5	5524.3	7710.9
VEHICLE MILES	30103.4	27931.5	26717.8	21880.4	14743.7	5797.8
CUMULATIVE GLOBAL COMPARISONS						
AVERAGE SPEED	21.5	18.6	15.4	9.6	3.1	1.3
VEHICLE TRIPS	27436.	25772.	25100.	23433.	17842.	12374.
VEHICLE HOURS	2919.5	3308.6	3807.8	5575.1	12247.7	17103.0
VEHICLE MILES	62635.1	61404.1	58672.5	53396.8	38134.7	22643.2

FIGURE 7 Summary statistics for Test Case 1.

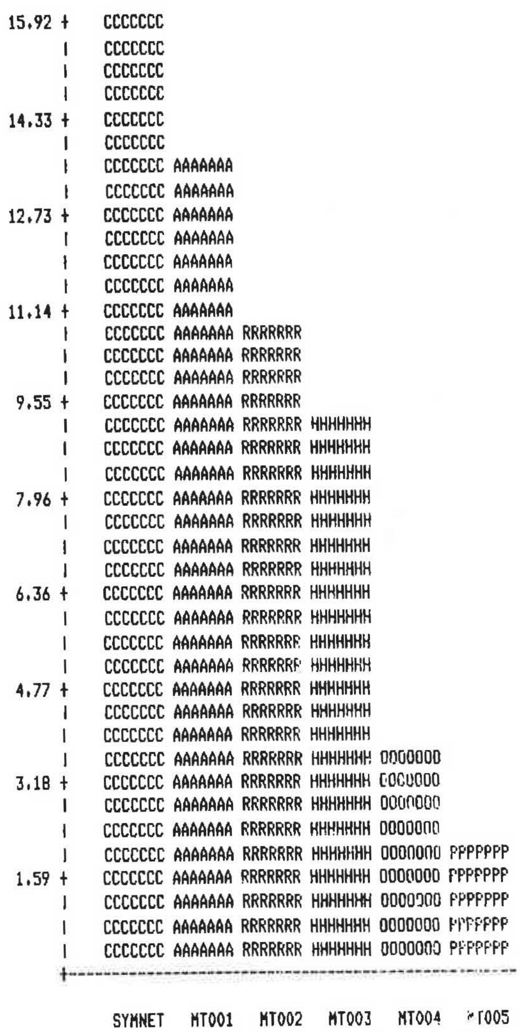

FIGURE 8 Average speed—arterial subnetwork.

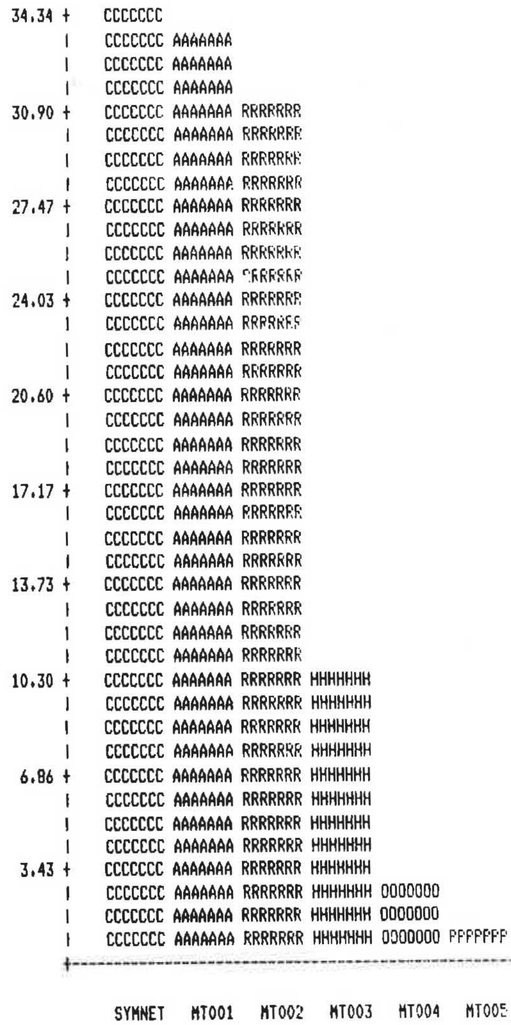

FIGURE 9 Average speed—freeway subnetwork.

	SYMNET	DT001	DT002	DT003	DT004	DT005	DT006
ARTERIAL SUBNETWORK							
AVERAGE SPEED	15.9	15.8	15.7	14.4	12.1	10.6	10.1
VEHICLE TRIPS	19747.	19686.	19221.	19310.	19008.	18496.	18153.
VEHICLE HOURS	2042.9	2048.4	2021.6	2211.6	2573.9	2834.9	2899.4
VEHICLE MILES	32531.7	32458.0	31828.9	31909.4	31212.5	29978.3	29345.0
FREEWAY SUBNETWORK							
AVERAGE SPEED	34.3	34.5	11.5	12.5	12.7	13.0	13.1
VEHICLE TRIPS	7689.	7418.	6153.	6263.	6327.	6352.	6350.
VEHICLE HOURS	876.6	855.0	2200.6	2012.8	1972.7	1889.0	1867.0
VEHICLE MILES	30103.4	29506.5	25226.1	25249.3	25054.4	24500.6	24378.6
CUMULATIVE GLOBAL COMPARISONS							
AVERAGE SPEED	21.5	21.3	13.5	13.5	12.4	11.5	11.3
VEHICLE TRIPS	27436.	27104.	25374.	25573.	25335.	24848.	24503.
VEHICLE HOURS	2919.5	2903.5	4222.2	4224.5	4546.5	4723.9	4766.4
VEHICLE MILES	62635.1	61964.5	57055.0	57158.7	56266.9	54478.9	53723.6

FIGURE 10 Summary statistics for Test Case 2.

```
INFORMATION SUMMARY FOR SCENARIO:DT002

        SCENARIO        ---> DT002
        DESCRIPTION     ----> SHORT TERM RECON, ONE DIRECTION:
                                                   LANE REDUCTIONS
        DATA CREATED    ---> 10 05 85
        USER NAME       ----> JOHN D. LEONARD
        AGENCY          ----> UC IRVINE -- ITS

CREATED FROM BASE NETWORK: SYMNET


LENGTH OF SIMULATION ( IN SECONDS ) = 3600

RECONSTRUCTION ZONE SPECIFIED AT LINK (  507, 508 )

                           SYMNET     DT002

NUMBER OF LANES      =         3         2
FREE FLOW  CAPACITY  =      2000      1400
SPEED THROUGH ZONE   =      33.8       1.7
TRIPS THROUGH ZONE   =    4496.0    2808.0
VOLUME/CAPACITY      =    0.7493    1.0029
MINUTES PER TRIP     =    0.8886   17.8684


RECONSTRUCTION ZONE SPECIFIED AT LINK (  508, 509 )

                           SYMNET     DT002

NUMBER OF LANES      =         3         2
FREE FLOW  CAPACITY  =      2000      1400
SPEED THROUGH ZONE   =      33.7       8.1
TRIPS THROUGH ZONE   =    4270.0    2596.0
VOLUME/CAPACITY      =    0.7117    0.9271
MINUTES PER TRIP     =    0.8906    3.6945


RECONSTRUCTION ZONE SPECIFIED AT LINK (  509, 510 )

                           SYMNET     DT002

NUMBER OF LANES      =         3         2
FREE FLOW  CAPACITY  =      2000      1400
SPEED THROUGH ZONE   =      34.6      16.9
TRIPS THROUGH ZONE   =    4463.0    2745.0
VOLUME/CAPACITY      =    0.7438    0.9804
MINUTES PER TRIP     =    0.8672    1.7786

DETOUR OPTION SELECTED    ----> SHORT-TERM USER-SPECIFIED

NUMBER OF DETOUR ROUTES  ---->   1
PERCENTAGE OF VEHICLES USING ROUTE  =    0

ORIGINAL ROUTE:
527   506   507   508   509   510   511

DETOUR   ROUTE:
527   506  7506    43    46    47    48    49    50    45  7511   511

               TRAVEL TIME COMPARISON ALONG ROUTES

                                 SYMNET     DT002

               ORIGINAL ROUTE  =    4.8      31.8
                 DETOUR ROUTE  =    9.8      11.4
```

FIGURE 11 Travel-time comparisons along detour route.

CONCLUSIONS

The CARHOP environment can provide the transportation planner and engineer with an effective method of measuring system performance during the reconstruction process. With CARHOP, assorted MOEs may be generated and used to evaluate alternative reconstruction strategies and possible mitigation procedures.

CARHOP provides an interactive, user-friendly, menu-driven, screen-oriented environment for the generation of reconstruction scenarios consisting of any combination of reconstruction zones (freeway lane constrictions) and detour strategies. CARHOP allows several scenarios to be created in a single interactive session, and the changes are stored for subsequent processing by the other modules of the CARHOP environment. Statistics output by CARHOP include vehicle speed, vehicle miles, vehicle trips, and vehicle minutes. They are compiled on a link-by-link basis and aggregated for the freeway and arterial subnetworks as well as for the network as a whole.

These outputs provide individual scenario reports on conditions in the immediate reconstruction zone. Histograms are generated that provide visual comparisons of various performance measures on each subnetwork and on the global network. These comparisons are intended to assist in the selection of reasonable freeway reconstruction and rehabilitation schedules.

To test and demonstrate the capabilities of CARHOP, a section of the Interstate 5 freeway in Orange County, California, was analyzed relative to various reconstruction strategies. The section analyzed, which begins at the interchange with State Route 55 and extends north to immediately south of the interchange with State Route 91, is scheduled for major reconstruction by the California Department of Transportation.

In the analysis, a number of reconstruction strategies, encompassing various diversion strategies involving detours along the surface street network, were evaluated. The results were useful in identifying courses of action that were optimal in the sense of traffic management and offered encouragement relative to the potential usefulness of this tool.

ACKNOWLEDGMENT

This work was supported, in part, by a contract with the California Department of Transportation and FHWA. Their support in this project is gratefully acknowledged.

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Commuter Perceptions of Traffic Congestion During the Reconstruction of I-45 North Freeway in Houston

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The I-45 North Freeway in Houston is currently undergoing a major reconstruction process. As Phase 2 of the reconstruction project is initiated, it has become necessary to implement several traffic-constricting activities, which are likely to affect mobility in an adverse manner. This paper presents an assessment of how the public is perceiving the traffic conditions during the reconstruction activities. Data on commuter travel times, travel distances, travel modes, and primary travel routes before and during reconstruction periods are presented. A similar effort was undertaken in Pittsburgh, Pennsylvania, during the reconstruction of the I-376 Parkway East. When possible, comparisons between the Houston and Pittsburgh data are presented. In general, it appears that most North Freeway commuters perceive little or no change in traffic conditions during this phase of the reconstruction. On the average, they depart 1.2 min earlier for work or school, travel distances measuring only 0.02 mi longer, and report a 1.2-min increase in travel time. Parkway East commuters, on the other hand, departed 29 min earlier for work or school, traveled distances 3.31 mi longer, and reported that it took 4 min longer to reach their destinations during reconstruction. Mode shifts in both Houston and Pittsburgh were very small, suggesting that the majority of commuters in both cities were not sufficiently inconvenienced by the reconstruction activities to look for another means of travel to work or school.

Phase 2 of a major reconstruction effort is currently under way along a 7.75-mi segment of the I-45 North Freeway in Houston, Texas. During this phase of reconstruction, an additional freeway mainlane will be added, and the transitway constructed for authorized high-occupancy vehicles will be enlarged to its final design width. This additional capacity will result in increased vehicle throughput during peak travel periods and better overall operation during off-peak hours.

As Phase 2 of the North Freeway reconstruction project is initiated, it has become necessary to temporarily implement several traffic-constricting activities, including the narrowing of freeway mainlanes, the closure of freeway ramps, and the periodic closure of freeway mainlanes during off-peak periods. These construction-related activities are likely to affect mobility in an adverse manner along the freeway mainlanes and throughout the entire North Freeway corridor. Of particular concern are the delays that may be incurred during the morning peak period when commuters are traveling to work or school. In order to minimize these potential delays, a traffic control plan has been developed by the Metropolitan Transit Authority of Harris County (METRO) and the Texas State Department of Highways and Public Transportation (SDHPT).

Texas Transportation Institute (TTI) is currently monitoring traffic conditions and collecting detailed and comprehensive data to evaluate the need for (and success of) various traffic control strategies during the North Freeway reconstruction process. In addition to travel time measurements, vehicle volume counts, and other types of measured data, TTI is also engaged in the assessment of how the public is perceiving traffic congestion and the special traffic control efforts. This assessment is being accomplished through the periodic distribution of questionnaires to a panel of North Freeway corridor commuters who have agreed to participate in the ongoing traffic congestion survey.

These perception studies are being undertaken as a result of past experiences showing that the public's perception of a project is often different from that indicated by objective studies. It is important to determine exactly how a project is being perceived by the public. If commuters do not perceive a need for (or benefit from) a particular traffic control effort, they are not likely to support its implementation (or continuation, or both), regardless of what objective studies may tell them. Such public support may be necessary when similar projects are considered in the future.

The first of the recurring North Freeway corridor commuter surveys was performed in May 1985 before Phase 2 of the mainlane reconstruction began. A second commuter survey was performed in December 1985 after Phase 2 reconstruction began. The results of the second survey are summarized in this paper. Many of the questions used in the North Freeway corridor commuter surveys are similar to those used in commuter surveys conducted before and during the reconstruction of the Parkway East (I-376) in Pittsburgh, Pennsylvania. When possible, for comparative purposes, the Pittsburgh data are also presented.

The primary purpose of the second commuter surveys in both Houston and Pittsburgh was to measure changes in departure times for work or school, travel times, travel distances, travel routes, and travel modes that have occurred since the reconstruction activities and traffic control strategies began. Specific activities and strategies in the North Freeway and Parkway East corridors are described in the following discussion.

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PARKWAY EAST AND NORTH FREEWAY RECONSTRUCTION ACTIVITIES

Reconstruction activities along the Parkway East corridor in Pittsburgh were extensive and the freeway peak-period capacity was reduced by approximately two-thirds for 16 out of 20 months of the reconstruction process. Traffic was restricted to one lane in each direction for months at a time. Entrance ramps at both ends of the reconstruction zone were restricted to use by high-occupancy vehicles. It has been estimated that 80,000 trips were restricted each day during the Parkway East reconstruction (*1, 2*).

In Houston at least three lanes of the North Freeway mainlanes remain open during the peak period for the peak direction of flow. Freeway capacity has been reduced by the narrowing of lanes and the placement of concrete median barriers adjacent to right-hand travel lanes. Entrance and exit ramps are periodically closed, but their use has not been restricted. No construction has been taking place within the transitway itself and transitway use has not been restricted during peak travel periods. Vehicle volume levels along the North Freeway mainlanes fluctuate less than 3 percent for the inbound, morning peak direction of flow. Volumes for the outbound direction during the afternoon peak period have increased from 4 to 10 percent (*1*).

TRAFFIC CONTROL STRATEGIES IN HOUSTON AND PITTSBURGH

Traffic control strategies implemented during the reconstruction of the Parkway East consisted of several alternative methods of improving people-moving capabilities. These included a new commuter train service, a third-party vanpool program, high-occupancy-vehicle ramps, new park-and-ride facilities, new express bus service, and traffic operations improvements on major alternative routes.

Implementation of such strategies for the North Freeway corridor in Houston has not been as extensive as was the case for Parkway East in Pittsburgh. Static signs have been placed along the freeway to encourage increased use of the existing, extensive park-and-ride and vanpool programs. No additional park-and-ride lots have been constructed because there is available capacity within the four existing lots that serve the North Freeway corridor. Capacity improvements to frontage road approaches at four interchanges have been developed and submitted to METRO by TTI. These temporary improvements will be implemented only if the freeway mainlane capacity is severely reduced or access restricted. No strategies similar to those used in Pittsburgh have been implemented thus far in Houston (*1*).

RESULTS OF NORTH FREEWAY COMMUTER SURVEY

Survey questionnaires were mailed to a total of 960 North Freeway commuters (395 transit users, 186 vanpool drivers, and 379 automobile commuters). Response rates ranged from 34 percent for the automobile commuter group to 39 percent for the transit user group to 65 percent for the vanpool driver group. The overall response rate was approximately 42 percent.

For analytical purposes, the results of the commuter survey were disaggregated into the following five groups:

- Local bus riders,
- Express bus riders,
- Park-and-ride users,
- Vanpool drivers, and
- Automobile commuters.

The express bus riders, park-and-ride users, and vanpool drivers surveyed typically utilize the North Freeway transitway, whereas the local bus riders and automobile commuters surveyed utilize the North Freeway mainlanes and frontage roads.

Official Work or School Start Times

North Freeway commuters were asked to list their official work or school start time. Median official work or school start times by commuter group are as follows:

Commuter Group	*Median Work or School Start Time (a.m.)*
Local bus riders	8:00
Express bus riders	7:00
Park-and-ride users	7:30
Vanpool drivers	7:30
Automobile commuters	7:00

The median work or school start time of 7:00 a.m. for both the automobile commuters and express bus riders is 30 min earlier than the median time of 7:30 a.m. for the park-and-ride users and vanpool drivers and 1 hr earlier than the median time of 8:00 a.m. for the local bus riders.

Departure Times

As indicated by the following median departure times, the reconstruction activities appear to be having little effect on the time that most commuters leave home for work or school:

	Median Departure Time (a.m.)	
Commuter Group	*Before Reconstruction*	*During Reconstruction*
Local bus riders	6:54	6:50
Express bus riders	6:00	6:00
Park-and-ride users	6:24	6:20
Vanpool drivers	6:20	6:25
Automobile commuters	6:15	6:15

In fact, the majority of commuters from all five survey groups report no change in their departure times since the reconstruction began (Table 1). Furthermore, 15 percent of the local bus riders, 10 percent of the express bus riders, 20

TABLE 1 CHANGES IN DEPARTURE TIMES FOR WORK OR SCHOOL

Departure Time During Reconstruction Compared to Before Reconstruction	Local Bus Riders	Express Bus Riders	Park-and-Ride Users	Vanpool Drivers	Auto Commuters
30 or more minutes earlier	--	5%	2%	--	4%
25 minutes earlier	--	--	1%	--	--
20 minutes earlier	--	--	1%	1%	2%
15 minutes earlier	--	5%	9%	1%	9%
10 minutes earlier	--	--	4%	3%	6%
5 minutes earlier	15%	--	4%	2%	4%
Same	70%	80%	59%	82%	67%
5 minutes later	15%	--	1%	4%	1%
10 minutes later	--	5%	5%	2%	1%
15 minutes later	--	5%	9%	2%	2%
20 minutes later	--	--	--	1%	--
25 minutes later	--	--	1%	--	--
30 or more minutes later	--	--	4%	2%	4%
Summary: Depart earlier	15%	10%	21%	7%	25%
Depart at same time	70%	80%	59%	82%	67%
Depart later	15%	10%	20%	11%	8%

percent of the park-and-ride users, 11 percent of the vanpool drivers, and 8 percent of the automobile commuters report that they are leaving home 5 to 35 min later during the reconstruction. On the other hand, sizable percentages of park-and-ride users and automobile commuters (21 and 25 percent, respectively) state that they are leaving home 5 to 45 min earlier during reconstruction.

Travel Times

Median travel times from home to work or school by survey group are as follows. Again, only slight variations have been occurring during reconstruction.

Commuter Group	Median Travel Time (min) Before Reconstruction	During Reconstruction
Local bus riders	30	40
Express bus riders	43	40
Park-and-ride users	45	45
Vanpool drivers	50	46
Automobile commuters	40	45

The majority of local bus riders, express bus riders, park-and-ride users, and vanpool drivers (76, 55, 56, and 67 percent, respectively) perceive that there has been no change in their travel times from home to work or school during reconstruction (Table 2). An additional 8 percent of the local bus riders, 30 percent of the express bus riders, 21 percent of the park-and-ride users, and 14 percent of the vanpool drivers indicate that travel times are actually 5 to 35 min shorter during reconstruction than before. Conversely, 16 percent of the local bus riders, 15 percent of the express bus riders, 23 percent of the park-and-ride users, and 19 percent of the vanpool drivers report longer travel times.

Responses from the automobile commuter group differ from those of the other four groups, however. Only 48 percent perceive no change and 10 percent perceive shorter travel times during reconstruction; 42 percent stated that travel times are 5 to 45 min longer.

Travel Distances

Median distances traveled from home to work or school ranged from 8 mi for the local bus riders to 30 mi for the vanpool drivers both before and during the reconstruction of the North Freeway.

Commuter Group	Median Travel Distance (mi) Before Reconstruction	During Reconstruction
Local bus riders	8	8
Express bus riders	23	23

TABLE 2 CHANGES IN TRAVEL TIMES FROM HOME TO WORK OR SCHOOL

Travel Time During Reconstruction Compared to Before Reconstruction	Local Bus Riders	Express Bus Riders	Park-and-Ride Users	Vanpool Drivers	Auto Commuters
30 or more minutes shorter	--	--	1%	3%	--
25 minutes shorter	--	--	1%	1%	1%
20 minutes shorter	8%	10%	2%	--	--
15 minutes shorter	--	5%	6%	4%	1%
10 minutes shorter	--	5%	8%	4%	2%
5 minutes shorter	--	10%	3%	3%	6%
Same	76%	55%	56%	67%	48%
5 minutes longer	8%	--	6%	9%	12%
10 minutes longer	--	10%	8%	8%	3%
15 minutes longer	--	--	7%	1%	12%
20 minutes longer	8%	--	1%	--	11%
25 minutes longer	--	--	--	--	3%
30 or more minutes longer	--	5%	1%	--	1%
Summary: Travel time is shorter	8%	30%	21%	14%	10%
Travel time is the same	76%	55%	56%	67%	48%
Travel time is longer	16%	15%	23%	19%	42%

Commuter Group	Median Travel Time (min) Before Reconstruction	During Reconstruction
Park-and-ride users	22	22
Vanpool drivers	30	30
Automobile commuters	24	24

At least 92 percent of all commuters surveyed report that the distance they travel from home to work or school had not changed since the North Freeway reconstruction began (Table 3); very small percentages indicate that their travel distances are somewhat shorter. On the other hand, approximately 5 percent of the vanpool drivers and 6 percent of the automobile commuters report that their travel distances are from 1 to 13 mi longer.

Primary Travel Routes

Automobile commuters and vanpool drivers were also asked to describe their primary travel routes from home to work or school before and during the North Freeway reconstruction. Their responses are given in Table 4, which indicates that only a small percentage of vanpool drivers and automobile commuters have varied their primary travel routes since Phase 2 of the reconstruction activities began. Generally speaking, the North Freeway has been the most heavily traveled route both before and during reconstruction. However, during reconstruction, use of the North Freeway by the vanpool drivers has increased 2 percent and use of the freeway by the automobile commuters has decreased 7 percent. The slight increase in vanpool use of the North Freeway may be due to the presence of the transitway, whereas the decrease in automobile commuter use may be due to their perception of worsening traffic congestion. (Primary travel routes for the local, express, and park-and-ride bus services have remained unchanged since the reconstruction activities began.)

Primary Travel Modes

The primary travel modes to work or school both before and during reconstruction are presented in Table 5, which shows that 10 percent of the express bus riders, 5 percent of the park-and-ride users, 6 percent of the vanpool drivers, and 6 percent of the automobile commuters during reconstruction had used different modes of transportation to work or school before reconstruction. Another item of interest is the large percentage of automobile commuters who report that they carpool. Approximately 55 percent of the automobile commuters carpooled before reconstruction and 53 percent are carpooling during reconstruction.

TABLE 3 CHANGES IN TRAVEL DISTANCES FROM HOME TO WORK OR SCHOOL

Travel Distance During Reconstruction Compared to Before Reconstruction	Local Bus Riders	Express Bus Riders	Park-and-Ride Users	Vanpool Drivers	Auto Commuters
8 miles shorter	--	--	--	--	1%
6 miles shorter	--	--	1%	--	--
3 miles shorter	8%	--	--	--	--
2 miles shorter	--	--	--	2%	--
1 mile shorter	--	--	--	--	1%
Same	92%	100%	97%	92%	92%
1 mile longer	--	--	1%	--	--
2 miles longer	--	--	--	2%	1%
3 miles longer	--	--	--	1%	1%
4 miles longer	--	--	--	--	1%
5 miles longer	--	--	1%	1%	1%
8 miles longer	--	--	--	--	1%
10 or more miles longer	--	--	--	2%	1%
Summary: Travel distance is shorter	8%	0%	1%	2%	2%
Travel distance is the same	92%	100%	97%	93%	92%
Travel distance is longer	0%	0%	2%	5%	6%

TABLE 4 PRIMARY TRAVEL ROUTES FROM HOME TO WORK OR SCHOOL

	Vanpool Drivers		Auto Commuters	
Primary Travel Route	Before Reconstruction	During Reconstruction	Before Reconstruction	During Reconstruction
N. Shepherd/N. Freeway	8%	7%	10%	10%
Airline	1%	---	---	2%
N. Freeway (mainlanes/AVL)	86%	88%	65%	58%
N. Freeway (frontage road)	---	1%	6%	6%
Hardy Road	1%	---	5%	5%
Eastex Freeway	1%	2%	---	1%
Crosstimbers/N. Freeway	---	---	1%	1%
Others	3%	2%	13%	17%

Information Concerning Reconstruction Activities

A final question asked of all five survey groups was "Do you think that the public has been kept adequately informed of the North Freeway reconstruction activities?" Their responses are given in Table 6. Between 23 and 40 percent of all commuters surveyed indicate "yes," whereas 25 to 41 percent respond "no" and 26 to 46 percent are unsure.

SUMMARY OF TRAVEL CHARACTERISTICS

Local Bus Riders

On the basis of the results of the second commuter survey, local bus riders typically left home at 6:54 a.m. before reconstruction and are leaving at 6:50 during reconstruction in order to get to work or school by 8:00 a.m. The local bus

TABLE 5 PRIMARY TRAVEL MODES TO WORK OR SCHOOL

Primary Travel Mode	Express Bus Riders		Park-and-Ride Users		Vanpool Drivers		Auto Commuters	
	Before Recon- struction	During Recon- struction	Before Recon- struction	During Recon- struction	Before Recon- struction	During Recon- struction	Before Recon- struction	During Recon- struction
Drive Alone	---	---	1%	---	2%	---	39%	47%
Carpool With Family	---	---	---	---	1%	---	6%	7%
Carpool With Others	10%	---	1%	---	1%	---	49%	46%
Vanpool	---	---	3%	---	94%	100%	4%	---
Bus	90%	100%	95%	100%	2%	---	2%	---

Note: 100% of the local bus riders reported that they commuted by bus both before and during reconstruction.

TABLE 6 IS PUBLIC KEPT ADEQUATELY INFORMED OF RECONSTRUCTION ACTIVITIES?

Adequate Information on Reconstruction Activities	Local Bus Riders	Express Bus Riders	Park-and-Ride Users	Vanpool Drivers	Auto Commuters
Yes	23%	40%	32%	38%	33%
No	31%	25%	35%	34%	41%
Not Sure	46%	35%	33%	28%	26%

riders traveled a distance of 8 mi in 30 min before reconstruction and 8 mi in 40 min (10 min slower) during reconstruction, indicating overall travel speeds of approximately 16 mph before reconstruction and 12 mph during reconstruction. None of the local bus riders have changed modes since the reconstruction began.

Express Bus Riders

Express bus riders typically left home at 6:00 a.m. both before and during reconstruction in order to get to work or school by 7:00 a.m. They reportedly traveled 23 mi in 43 min before reconstruction and 23 mi in 40 min during reconstruction. This indicates they averaged 32 mph before reconstruction and almost 35 mph during reconstruction. About 10 percent of the express bus riders had previously carpooled before reconstruction.

Park-and-Ride Users

Park-and-ride users typically left home at 6:24 a.m. before reconstruction and 6:20 a.m. (4 min earlier) during reconstruction in order to arrive at work or school by 7:30 a.m. They typically traveled a distance of 22 mi in 45 min both before and during reconstruction, indicating an overall travel speed of 29 mph. About 3 percent of the park-and-ride users had previously vanpooled before reconstruction; 1 percent drove alone and 1 percent carpooled with others than family members.

Vanpool Drivers

Vanpool drivers normally left home at 6:20 a.m. before reconstruction and 6:25 a.m. (5 min later) during reconstruction in order to arrive at work locations by 7:30 a.m.

They reportedly traveled 30 mi in 50 min before reconstruction and 30 mi in 46 min (4 min faster) during reconstruction. Thus they appear to have averaged 36 mph before reconstruction and 39 mph during reconstruction. Approximately 2 percent of the vanpool drivers were previously bus riders, 2 percent drove alone, and 2 percent carpooled with family or others before reconstruction.

Automobile Commuters

Automobile commuters surveyed reported earlier departure and work start times than the other four survey groups. In general, automobile commuters left home at 6:15 a.m. both before and during reconstruction in order to arrive at work by 7:00 a.m. They traveled a median distance of 22 mi both before and during reconstruction. Travel time of 45 min during reconstruction is 5 min slower than before reconstruction. Thus, they averaged 33 mph before reconstruction, but only 29 mph during reconstruction. Approximately 4 percent of the automobile commuters had previously vanpooled before reconstruction and 2 percent had made the trip by bus.

COMPARISON OF HOUSTON AND PITTSBURGH SURVEY DATA

Median departure times from home to work or school both before and during reconstruction for the North Freeway corridor commuters (all modes) in Houston and the Parkway East corridor commuters (all modes) are given in Table 7. Median trip travel times and distances for both survey groups are given in Tables 8 and 9, respectively. Generally speaking, North Freeway corridor commuters leave home earlier, travel longer distances, and take more time to reach work or school locations than Parkway East corridor commuters (both before and during reconstruction periods).

Average Changes: Before and During Reconstruction

Looking at the average changes that took place in Pittsburgh and Houston reveals the following:

- Parkway East commuters departed for work or school 29 min earlier during reconstruction, whereas North Freeway commuters depart only 1.2 min earlier.
- Parkway East commuters traveled distances 3.31 mi longer during reconstruction, whereas North Freeway commuters travel distances only 0.02 mi longer.
- Parkway East commuters reported that it took 4 min longer to travel to work or school during reconstruction, whereas North Freeway commuters reported that it takes only 1.2 min longer.

Modal Split Data: Before and During Reconstruction

Modal split data for the North Freeway corridor commuters and the Parkway East commuters are given in Table 10, which indicates that vanpoolers make up a much larger percentage of the commuter group in Houston than in Pittsburgh. This is to be expected, because vanpooling has long been a popular travel mode in Houston, whereas vanpooling programs were just being initiated in Pittsburgh.

Table 10 also indicates very small modal shifts in Houston and Pittsburgh during the reconstruction activities. This would suggest that the majority of commuters in both cities were not sufficiently inconvenienced by the reconstruction activities to look for other means of travel to work or school.

TABLE 7 DEPARTURE TIMES FOR WORK OR SCHOOL BEFORE AND DURING RECONSTRUCTION PERIODS IN HOUSTON AND PITTSBURGH

	North Freeway - Houston		Parkway East - Pittsburgh	
Departure Time from Home	Before Reconstruction	During Reconstruction	Before Reconstruction	During Reconstruction
Before 6:00 a.m.	15%	16%	6%	8%
6:00 - 6:30 a.m.	53%	51%	10%	13%
6:31 - 7:00 a.m.	19%	19%	45%	44%
7:01 - 7:30 a.m.	11%	12%	38%	33%
7:31 - 8:00 a.m.	1%	1%	1%	2%
After 8:00 a.m.	1%	1%	---	---
Average Change: During-Before	1 minute earlier		29 minutes earlier	

Source: December 1985 Houston North Freeway corridor commuter surveys and Reference 2.

TABLE 8 COMPARISON OF WORK AND SCHOOL TRIP TRAVEL TIMES BEFORE AND DURING RECONSTRUCTION PERIODS IN HOUSTON AND PITTSBURGH

Trip Time Distribution	North Freeway - Houston		Parkway East - Pittsburgh	
	Before Reconstruction	During Reconstruction	Before Reconstruction	During Reconstruction
1-10 minutes	2%	1%	2%	1%
11-20 minutes	5%	5%	24%	10%
21-30 minutes	14%	13%	28%	24%
21-40 minutes	23%	23%	17%	23%
41-50 minutes	34%	33%	28%	23%
>50 minutes	22%	25%	11%	20%
Average change: During-Before	1.2 minutes longer		4 minutes longer	

Source: December 1985 Houston North Freeway Corridor commuter surveys and Reference 2.

TABLE 9 COMPARISON OF WORK AND SCHOOL TRIP TRAVEL DISTANCES BEFORE AND DURING RECONSTRUCTION PERIODS IN HOUSTON AND PITTSBURGH

Trip Distance Distribution	North Freeway - Houston		Parkway East - Pittsburgh	
	Before Reconstruction	During Reconstruction	Before Reconstruction	During Reconstruction
1-5 miles	4%	3%	6%	5%
6-10 miles	5%	6%	32%	29%
11-15 miles	9%	8%	28%	30%
16-20 miles	12%	13%	20%	20%
21-25 miles	36%	36%	11%	11%
>25 miles	34%	34%	4%	5%
Average change: During-Before	0.2 miles longer		3.31 miles longer	

Source: December 1985 Houston North Freeway corridor commuter surveys and Reference 2.

TABLE 10 MODAL SPLIT FOR COMMUTERS SURVEYED BEFORE AND DURING RECONSTRUCTION IN HOUSTON AND PITTSBURGH

Primary Travel Mode	North Freeway - Houston		Parkway East - Pittsburgh	
	Before Reconstruction	During Reconstruction	Before Reconstruction	During Reconstruction
Drove Alone	13%	15%	37%	34%
Carpooled with Family	2%	2%	9%	9%
Carpooled with Others	17%	15%	19%	20%
Vanpooled	30%	30%	3%	5%
Transit	38%	38%	31%	31%
Other	---	---	1%	1%

Source: December 1985 Houston North Freeway corridor commuter surveys and Reference 2.

(The small modal shifts that occurred in Pittsburgh were particularly disappointing considering the new commuter rail, park-and-ride service, express bus service, and vanpool programs that were implemented as traffic control strategies.)

COMPARISON OF SURVEY RESULTS WITH FIELD MEASUREMENTS

The commuter surveys in Houston and Pittsburgh were undertaken to identify and measure changes in travel behavior during the reconstruction periods. In addition to these surveys, a variety of field measurements were also performed along the North Freeway and Parkway East corridors. Comparisons between the survey responses and field measurements were possible in several areas. In general, there was a high degree of consistency between the field measurements and the commuter survey responses in both Houston and Pittsburgh.

Departure Time Changes

In Houston the majority of commuters report no change in their departure times from home to work or school since the reconstruction began. Vehicle volume counts along the North Freeway corridor indicate that there has been no shift in the time that the a.m. peak period occurs (*1*).

Survey responses in Pittsburgh indicated that there was a general shift toward earlier departure times during reconstruction activities. Volume counts within the Parkway East corridor indicated that the peak travel period shifted approximately 30 min earlier during the reconstruction period, which is consistent with average change in reported departure times (*2*).

Travel Time Changes

The majority of North Freeway corridor commuters in Houston perceive that there has been little or no change in their travel times from home to work or school during reconstruction. The average change in a.m. travel time as reported by all modes was 1.2 min longer during reconstruction. Results of travel time and delay studies along the North Freeway indicate that during reconstruction, the average travel times in the a.m. peak decreased by 0.1, 1.9, and 0.9 min, respectively, for trips beginning at 6:30, 7:30, and 8:30 a.m. (*1*).

In Pittsburgh survey responses from the Parkway East commuter panel indicated an increase of about 5 min for work or school trips during the reconstruction period; trip time measurements indicated an average travel time increase of about 6 min for the morning peak (*2*).

Primary Travel Route Changes

Only a very small percentage of the North Freeway commuters in Houston report to have varied their primary travel routes since the reconstruction began. Generally speaking, the North Freeway has been the most heavily traveled route both before and during reconstruction. Volume levels recorded along the freeway mainlanes have changed by less than 3 percent for the inbound flow. No increase in transitway use has been observed (*1*).

Volume levels during reconstruction of the Parkway East in Pittsburgh decreased by slightly more than half. Survey responses from the commuter panel indicate that 40 percent of the work trips formerly using the Parkway East diverted to other routes (*2*).

CONCLUSION

From the results of the Houston survey, it appears that most commuters have perceived no change in traffic conditions along the North Freeway during reconstruction. In fact, 70 percent report that they leave home at the same time, 94 percent travel the same distance, and 57 percent report no change in the length of time it takes to travel to work or school. An additional 12 percent leave home later, 5 percent travel shorter distances, and 28 percent take less time to get to work or school during reconstruction. This would indicate that the majority of commuters perceive that (so far) they have not been significantly affected by this phase of the reconstruction process. Indeed, several commuters commented that, considering the magnitude of the project, disruption to traffic has been minimal. However, as construction sequences begin to directly affect the freeway mainlanes, additional delay could occur, travel patterns may be altered, and additional traffic control strategies may be warranted.

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Mitigating Corridor Travel Impacts During Reconstruction: An Overview of Literature, Experiences, and Current Research

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The impacts of major urban transportation construction projects on existing traffic patterns and economic activity are undeniable and significant. Reducing the costs of delay to the local community of users and nonusers imposed by highway and bridge reconstruction projects requires that appropriate management actions be taken. To mitigate these external costs, strategically planned actions have been adopted or are being considered in the execution of several recent or planned projects to reduce construction-related impacts. Examples of such actions include (a) innovative project-scheduling strategies; (b) new construction techniques, including the use of special or prefabricated materials; (c) use of contract incentives to encourage more timely performance; (d) construction and use of temporary traffic lanes, (e) traffic improvements to alternative routes, (f) increased supply of public transportation services, (g) private and public promotion of ridesharing, and (h) public awareness campaigns via printed materials and the news media. A survey and review are given of state-of-the-art reconstruction techniques, traffic accommodation strategies, construction quality-control measures, and project development and evaluation processes as they have been applied to mitigate corridor travel impacts during reconstruction projects.

The impacts of major urban transportation construction projects on existing traffic patterns and economic activity are undeniable and significant. Reducing the costs of delay to the local community of users and nonusers imposed by highway reconstruction projects requires appropriate management actions. To mitigate external costs, strategically planned actions have been adopted or are being considered to reduce construction-related impacts. Examples of such actions include

- Innovative scheduling of construction activities to maintain traffic flow on the maximum possible number of lanes during peak periods;
- Introduction of new materials or placement techniques to speed construction (e.g., prefabricated elements, temporary load-bearing spans, quick-curing concretes);
- Contract incentives to encourage on-time performance with work conducted in a manner least disruptive to existing traffic patterns:
- Construction of temporary lanes or ramps, which can often become the shoulders or emergency pull-off areas of the completed project;
- Implementation of alternative transportation strategies in the affected corridor, such as ridesharing promotions, special parking arrangements, or additional transit services;
- Traffic flow improvements to alternative routes, such as parking restrictions along curb lanes, turning restrictions, intersection improvements, and the retiming of traffic signals; and
- Use of media advertising and public-private cooperation to inform the public of how to collectively make the best of a difficult situation.

The primary purpose of each of these efforts, whether it be an innovation in construction technology or a creative people-moving strategy, is to reduce the external costs of a reconstruction project. The objectives of the research described in this paper are

1. To investigate and document the critical interrelationships among state-of-the-art reconstruction techniques, traffic accommodation strategies, construction quality issues, and the project development and evaluation process, and
2. To formulate and document a Corridor Reconstruction Project Evaluation Process (CRPEP) based on the foregoing investigations.

Specialty conferences on traffic management strategies for special events were held before the 1985 and 1986 Transportation Research Board Annual Meetings at which pertinent experience was presented and discussed (*1-3*). Many articles in recent issues of the ASCE *Transportation Engineering Journal* and the *Transportation Research Record* focus on traffic management strategies in and around work zones (*4-6*). However, a great deal of additional documentation and project data is available from the agencies responsible for these projects.

There are several issues of particular concern in preparing for the construction phase impacts of a transportation

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project. One is the explicit trade-off between reducing direct project costs and mitigating the external or indirect costs to travelers and economic activity. The best action to minimize direct project costs would be to close off the work zone entirely so that construction crews can perform their work unaffected by passing traffic and travelers are not exposed to construction work-zone hazards. However, the complete shutdown of a facility during reconstruction is often not a reasonable alternative in heavily traveled corridors where an inadequate level of service is available from alternative routes or modes to accommodate the shift in travel volumes.

Because the implementation of special work schedules and traffic accommodation strategies increases the project's costs in attempts to lower external costs, the evaluation of travel characteristics and transportation facilities in the area surrounding the work zone is the key to the proper assessment of the trade-off between direct and indirect project costs. Moreover, mitigation strategies can be as important to less heavily used facilities as to projects involving high traffic volumes. For example, preconstruction planning for a moderately traveled facility, with few alternative routes or modes of travel in the neighboring area, may reveal potentially greater travel impacts than the reconstruction of a major expressway for which there are several alternative routes that can each handle a portion of the diverted traffic volumes. This is just one illustration of how several interrelated factors must be taken into account when a reconstruction project is planned and managed. A prudent analysis of such factors, with input from local communities, can be used by engineers and planners to develop appropriate types and levels of innovative transportation and construction management strategies that a particular project warrants.

Traffic management strategies for one major reconstruction project (I-376) have been evaluated and documented in detail (*7*). Procedures for planning and implementing reconstruction traffic management schemes were suggested in that study and are being adopted by other projects around the country (*8*). However, steps by which alternative project strategies can be developed, assessed, and selected for implementation are not currently documented as a systematic procedure involving all of the cost and quality control aspects of corridor traffic management. Moreover, many different public and private organizations are involved in the construction planning and management process, and responsibilities are not always clearly defined for assessing the effectiveness of alternative plans and deciding on the best set of strategies to adopt. Several recent projects discussed next serve to characterize the types of innovative strategies and project-planning procedures that are of key relevance to these issues.

SOME RECENT EXPERIENCES

There are several Interstate reconstruction projects in various stages of planning and construction that serve as useful and pertinent examples because of the data and experiences that can be obtained from them. Reconstruction of Chicago's Edens Expressway (I-94), 1978-1980, and reconstruction of Pittsburgh's Parkway East (I-376), 1981-1982, are two examples of completed projects for which transportation management procedures were extensively planned, monitored, and documented. In addition, Leisch and Associates (*9*) compiled case studies of the planning and design features of several past or ongoing freeway reconstruction projects as of 1983.

The Edens Expressway project might be considered as an early example of a major reconstruction of a heavily traveled urban Interstate freeway. For this project, the Illinois Division of Highways developed six alternative traffic control plans and evaluated them on the basis of 12 primary considerations (*10*). The social and environmental impacts of that project, grouped into six basic categories, have also been documented (*11*). In part, experiences with the Edens project alerted federal, state, and local transportation officials that the impacts of Interstate reconstruction projects in urban areas would indeed be significant, but that these impacts could be mitigated by means of an effective transportation management plan and innovative construction scheduling (*12*).

To mitigate impacts of the Parkway East (I-376) reconstruction project, the Pennsylvania Department of Transportation (PennDOT) implemented an experimental plan of people-moving strategies that included a third-party vanpool program, new express bus service, high-occupancy-vehicle ramps for passage through the work zone, new park-and-ride lots, a new commuter rail service, and traffic flow improvements to many local arterials (*13, 14*). This experiment tested the concept of implementing alternative transportation strategies throughout a travel corridor in order to reduce the construction-related impacts of a single major project. A research team from GAI Consultants and Carnegie-Mellon University (CMU) conducted extensive data-gathering surveys before, during, and after this project in order to measure the relative effectiveness of these various strategies (*7, 15, 16*). As an immediate benefit of the GAI-CMU evaluation, some of the people-moving strategies and traffic operations plans were modified to be more cost-effective in the second year of reconstruction on the basis of data collected during the first year, and additional low-cost traffic control measures were implemented to reduce traffic delay (*17*).

OTHER PLANNED OR CURRENT PROJECTS

Experiences with transportation management plans for the completed reconstruction projects cited in the previous section make up a small subset of those available for case studies. These projects are representative of significant innovative approaches to reconstruction, and the lessons learned from them are being applied to current projects elsewhere. The research described in this paper focuses on these and other major planned and current reconstruction projects, the most significant of which are as follows:

Seattle (I-5)	Syracuse (I-81)
Hartford (I-91)	Madison (I-90/94)
New Jersey (I-287)	Allentown (I-78)
Pittsburgh (I-376)	Philadelphia (I-76)
Ft. Lauderdale (I-95)	Minneapolis (I-394)
Boston (I-93 SE Expressway)	Atlanta Freeway System
Detroit (Lodge Freeway)	Los Angeles (Santa Ana Frwy)
Portland (Banfield Light Rail/Freeway Project)	Houston (North and Southwest Freeways)

Maryland (Woodrow Wilson and Cabin John bridges)	District of Columbia (George Mason Bridge)
Chicago (Edens, Eisenhower, and Dan Ryan expressways)	Dallas (North Central Project)
	San Antonio (I-35)

Several feature articles in both journals and trade magazines highlight the special aspects of several of the foregoing projects (*17-22*). A primary product of the initial research for this study was the compilation of an annotated bibliography containing abstracts or summaries of over 100 articles, reports, and other pertinent reference material (*23*). At the same time, a "short list" was developed of case study projects that were found to represent the greatest diversity of innovative experiences, of which there seemed to be ample documentation. The screening process began with the development of a "long list" of case study candidates, such as those shown earlier, that were found through the literature search and through discussions with FHWA officials. As candidate projects were identified, the criteria that were eventually used to select a smaller number of case study projects to be examined in further detail were refined. These selection criteria or project characteristics were grouped into the following five basic categories:

1. Reconstruction techniques and scheduling
   a. Pavements
      (1) Rigid
      (2) Flexible
      (3) Composite
   b. Bridge decks
      (1) Modular/precast
      (2) Cast-in-place
      (3) Metallic
   c. Project scheduling
      (1) Accelerated schedules
      (2) Night closure of lanes
      (3) Special staging areas
2. Traffic accommodation strategies
   a. Work-zone traffic control
   b. Lane-shift decisions
   c. Modal shifts
   d. High-occupancy-vehicle (HOV) measures
3. Travel impact forecasting procedures
   a. Sketch-planning techniques
   b. Computer simulation models
4. Construction quality issues
   a. Effects of rapid installation
   b. Performance of special materials
   c. Sampling and testing procedure
5. Project management and evaluation procedures
   a. Value engineering
   b. Use of incentive clauses
   c. Traffic management teams
   d. Contract administration

Reconstruction projects vary greatly in the extent to which special construction techniques or management strategies were employed. Certain projects, such as I-376, demonstrate a great variety of alternative travel strategies, whereas projects such as the Woodrow Wilson Bridge demonstrate uses of innovative construction methods or organizational control. Consequently, the experiences of many different projects must be taken as a cross section in order to document the relationships among the many diverse project attributes, not all of which are present or encountered on any one project. In the selection of projects to examine in depth as case studies, diversity was emphasized in the following areas:

1. Project location (geographical, rural, urban, suburban),
2. Facility type (road, bridge, tunnel, number of lanes),
3. Project type (reconstruction, redecking, major overlay),
4. Methods and materials (prefabricated slabs, segmental versus continuous pours, polymer concretes, steel decking),
5. Traffic handling requirements (high versus low volumes, critical peaks),
6. Project monitoring and evaluation procedures, and
7. Construction quality control measures.

CHARACTERISTICS OF ROAD RECONSTRUCTION PROJECTS

Reconstruction Techniques and Scheduling

A number of authors [e.g., Grimsley and Morris (*24*)] have documented the successful use of special methods and materials for highway and bridge rehabilitation and reconstruction projects. As one example, the use of precast or preconstructed bridge sections has reduced the time a bridge must remain out of service during reconstruction. A rather recent and very dramatic example of the degree to which innovative construction techniques and scheduling can be designed and used to reduce traffic disruptions is the Woodrow Wilson Bridge (I-95) redecking project (*25, 26*). The project incorporated innovations in traffic maintenance, design of the replacement sytems, applications of polymer concrete, and coordination of construction activities.

The technical and traffic management aspects of this project were keyed to the special conditions of the project. The entire redecking was accomplished through the use of prefabricated slabs lifted into place from a barge moored below the work site. Whereas traffic was restricted during all construction periods of the I-94 and I-376 projects, the contractor for the Woodrow Wilson Bridge project was able to keep all three lanes open in each direction across the bridge during all peak-period hours. This provision of peak-period capacity was made possible through the use of steel grid decks that temporarily spanned the current segment under construction each day. In addition, a rapid-curing polymer concrete was used for cast-in-place bearing pads that achieved their full load-bearing strength less than 24 hr after being poured. The entire project exemplified how the use of special construction materials, techniques, and scheduling can result in the reconstruction of a critical link without excessive disruptions to existing travel.

Other articles or reports dealing with the use of special materials and placement techniques on reconstruction projects include that by Meyer et al. (*27*). Conducting reconstruction and maintenance activities at night has become a common phenomenon in urban areas where daytime vehicle volumes mandate that facilities operate at normal capacity. Research suggests that nighttime work is not only feasible, but that it is also the most practical and cost-effective schedule in many cases (*28, 29*).

Traffic Accommodation Strategies

Although one part of this research is primarily an investigation of case study projects involving innovative reconstruction techniques, another major part is the investigation of cost-effective traffic accommodation strategies for implementation in either the work zone or the affected corridor. Traditional maintenance and protection-of-traffic plans focus primarily on the work zone itself. However, as travel accommodation strategies become more innovative, they often entail measures that fall both within and outside of the work zone. For example, HOV ramps that allow for priority passage of carpools, vanpools, and buses through a work zone can be implemented in conjunction with strategies outside the work zone, such as ridesharing incentives and special bus services that utilize these ramps.

A survey of recent publications revealed a manual prepared by the New York State Department of Transportation (*8*). The major emphasis of this manual is specific transportation system management (TSM) strategies that have been used or may be utilized in traffic management efforts. This manual draws heavily on the experiences of applying TSM strategies in Pittsburgh, Syracuse, and Boston in their respective reconstruction projects. TSM strategies that are discussed in this manual include HOV actions, additional bus transit, park-and-ride lots, vanpooling, commuter rail service, ferry service, expansion of alternative routes, and the use of public information (e.g., newspapers and public advertisement campaigns) as a way of handling traffic in reconstruction corridors. Each chapter of this manual gives information on a different TSM strategy and presents the following: (a) where the TSM action was implemented and (b) a description of the specific program and how it was incorporated in the overall TSM plan. Suggestions about the effectiveness of each particular TSM strategy are included.

Another document describing TSM strategies as they can be applied to travel impact mitigation is NCHRP Report 263 (*30*), which is written in the form of a user's manual and is supplemented by "training aids" consisting of audiovisual slide/tape modules and interactive computer-assisted instructions (*31*). This set of reports and aids is designed to assist project personnel in the planning, design, and implementation phases of all types of low-cost TSM improvements and will assist agencies in applying the approach to identify feasible, workable, and low-cost solutions to corridor transportation. They present the research findings in a form directly applicable by transportation professionals at municipal, regional, and state agencies.

The major emphasis of past and recent research in the area of traffic control during highway maintenance, rehabilitation, and reconstruction activities has been focused on work-zone traffic control rather than corridorwide travel impact mitigation. In particular, there are three key documents describing work-zone traffic control (*32-34*). Several recent papers have also been either presented or published on this topic (*2, 35-40*). Other papers and reports (*29, 41*) deal with the cost and safety aspects of work-zone traffic control. Finally, several researchers from both the United States and abroad have developed computer simulation models of work zones and temporary lane closures to predict the responses of traffic flows to changes in the supply characteristics of the roadway (*42-44*).

Traffic accommodation strategies both within and outside the work zone have interrelated impacts on mode shifts, route shifts, and changes in travel demand. Mode shifts include changes from low-occupancy vehicles to high-occupancy vehicles (public or private). Route shifts constitute the largest change in travel patterns during a reconstruction project, as is evident from several studies (*7, 11, 45*). However, travel demand changes other than mode and route shifts may also take place to a limited extent. Examples of travel demand changes are reductions in trip making, trip chaining, changes in departure times, or changes in destination choice. In comparing departure times from home before and during the Parkway East project, it was found that the average commuter departed 19 min earlier during construction in anticipation of construction delays (*7*). Stores and restaurants in downtown Pittsburgh also reported a significant drop in sales, indicating a shift in destination choices or trip-making frequencies of discretionary trips. Another working paper from this research will describe the following characteristics of alternative traffic accommodation strategies:

1. Location of primary focus (work zone versus corridor);
2. Responsibility for implementation (state transportation department, municipal traffic department, state or municipal police, metropolitan planning organization, regional transit authority, contractor, or project manager);
3. Implementation requirements (costs, lead time, materials, personnel, organizational coordination, media advertising);
4. Impacts on mode shifts, route shifts, and travel demand; and
5. Flexibility, that is, its ability to be adjusted to project needs.

The last aspect of a travel accommodation strategy is particularly important to the minimization of project risk, as it is for all aspects of a project's management plan. Special people-moving strategies that require a large initial investment to put in place carry with them a high risk of not being cost-effective. On the other hand, adjustments to bus services or adding cars to an existing commuter train have a much lower initial cost and can be revised according to traveler responses. In either case, it is important to estimate the costs of traveler impacts so as to make the proper trade-off (*46*).

In addition to the description of traffic accommodation strategies, later research will document the experiences of projects in which some of these strategies were employed. The Parkway East project represents the greatest number of travel accommodation strategies used for any single project. Because strategies focused outside the work zone are now only in the design or early construction phase in other projects (e.g., Minneapolis, I-394; New Jersey, I-287: Chicago, Dan Ryan Expressway; Houston, Southwest Freeway), the extent to which actual construction experience with these strategies is documented is somewhat limited.

Construction Quality Control Issues

Another area being investigated is the construction quality control issues that arise as new materials, methods, and scheduling strategies are introduced to the construction process. Of specific concern to this investigation will be

1. Strength or curing characteristics of new materials, or both;
2. Differences in road surface characteristics and structural integrity of segmental versus continuous construction and between prefabricated versus pour-in-place construction effects of traffic vibrations on the curing of materials;
3. Effects of traffic vibrations on the curing of materials;
4. Effects of traffic accommodation strategies on the abilities of workers to operate machinery and perform different tasks;
5. Quality difference between daytime and nighttime work;
6. Changes in workmanship when staffing requirements place excessive demands on the available labor supply;
7. Changes in quality due to accelerated schedules; and
8. Effects on quality of less frequent inspections.

Rapid installation and the effects of traffic-induced vibrations are two topics for which reports of recent research are readily available (*47*). Several articles in *Quality in the Constructed Project* [the proceedings of an ASCE workshop (*48*)] make it apparent that measuring "quality" is often a very difficult charge (*49*). An inspector can examine material placement, check for apparent flaws, or observe that pour is performed properly, but in-service quality deficiencies may not be obvious until some time later. As projects become more organizationally complex, with both owners and contractors appointing supervisory personnel to a project site, the issues of quality control and accountability also become less clearly defined (*50*). This indicates that it may be difficult to find existing documentation on the degree to which quality control problems have arisen on particular projects. However, the extent to which quality control procedures and guidelines are prescribed for road and bridge projects of various types can be obtained from the responsible project agencies.

Two documents in particular have been published by FHWA on quality control during construction (*51, 52*). The first of these emphasizes statistical concepts and techniques as they can be applied to quality assurance in general and specifically to construction materials. The second document cited is a more management-oriented description of how to implement quality assurance programs at the local level for construction and maintenance activities. A third document goes a step further to examine the cost-effectiveness of alternative sampling and testing programs in paving construction (*53*).

An issue related to quality assurance that is of vital public concern in nearly all professional fields today is that of liability, both with regard to personal injuries and contract disputes. The safety of workers, travelers, and pedestrians in and around the construction zone as well as the effect of diverted traffic volumes on the safety of travelers and pedestrians along more heavily traveled alternative routes are issues that local officials will be confronted with when planning for reconstruction projects (*54-56*). In addition, contract claims over work-order changes, delays, pay schedules, and unexpected conditions are project risks that require prudent administration and planning on the part of all the public and private officials involved (*57, 58*). On the Woodrow Wilson Bridge project, the Maryland Department of Transportation entered into an agreement with the contractor that every change of work request would be acted on within a 24-hr period so as not to delay the project.

Project Management and Evaluation Procedures

Project evaluation procedures can be effectively applied both before and during a reconstruction project in order to increase the cost-effectiveness of organizational management schemes being planned or implemented or in place. Many of the tools that can be used to achieve greater effectiveness can be found in the literature on value engineering (*59*). Two reports give examples of value engineering concepts applied specifically to highway reconstruction projects (*60, 61*).

Another important approach being successfully employed in the management of local transportation systems, both on a continual basis and during times of special need such as major reconstruction, is the development of traffic management teams, regularly scheduled meetings of planners, engineers, consultants, police officers, and transportation agency and local government officials, each of whom has a different perspective and primary concern with regard to the manner in which a project and its impacts are handled. Documentation of this approach, its advantages, and its disadvantages can be found elsewhere (*62-65*).

As another example of the corridor management team concept in practice, the Parkway East project (*7*) was quite more than an experiment with innovative transportation strategies. One experience that occurred on this project (*66*) was the manner in which travel impact mitigation strategies were modified to be more cost-effective for the second year of reconstruction. This indicates a significant amount of communication and cooperation between those monitoring the strategies and those in charge of their implementation. Public acceptance and utilization of these strategies and respect for the massive coordination effort at hand were due in large part to the cooperation between public agencies and private firms involved.

Other examples of actions that played an important role in the success of the Parkway East project are the following. A public media campaign staged by local radio and TV stations effectively diverted many travelers to alternative routes and modes even before the project began. The Southwestern Pennsylvania Regional Planning Commission (SPRPC) continued to encourage ridesharing as it had been doing for several years, and it also worked closely with Van Pool Services, Inc., to promote vanpooling in the affected corridor. Carpools and vanpools were allowed to enter the work zone via HOV ramps that were monitored 14 hr each weekday by Pittsburgh police. SPRPC provided data that assisted the project monitoring team to establish screenline count locations.

Also for that project, the Port Authority Transit of Allegheny County provided route, run, patronage, and cost data for the special express bus services. SPRPC and Van Pool Services also provided data to researchers at Carnegie-Mellon University that were used in subsequent modeling studies to forecast vanpool formations and alternative route volumes (*42, 67, 68*). The reconstruction contract also provided for traffic police to be stationed at 21 locations in the affected corridor, most of which were outside of the actual work zone. In short, the spirit of public-private cooperation exhibited during the Parkway East project was itself a key element to its success.

Contract administration plays an important role in the on-time performance of a construction project. Recent experiences have shown, in fact, that reasonable incentives can create such significant productivity improvements that the total cost of the project (i.e., public tax dollars plus the public cost of traffic disruptions) is lowered. The most comprehensive coverage of recent experiences and research concerning the impact of incentive-disincentive clauses on contract performance was produced by Viljoen as a Ph.D. thesis (*69*). Other brief articles include those by Officer (*70*) and by Weed (*71*).

CORRIDOR RECONSTRUCTION PROJECT EVALUATION PROCESS

Anderson and Hendrickson describe a reconstruction traffic mitigation planning procedure developed by GAI Consultants, Inc., and Carnegie-Mellon University in 1982 during reconstruction of the Parkway East (*7*). Procedures employed in other traffic management plans that have been examined can be categorized according to the following attributes:

1. Current practice,
2. Recent developments,
3. Value engineering approaches,
4. Use of contract incentives and disincentives,
5. Use of computer models or sketch-planning techniques,
6. Use of management information systems or database software for project monitoring and control, and
7. Contract administration procedures for shop drawing reviews, material approvals, field change approvals, or contract time calculations.

GAI Consultants, Inc., prepared the traffic management plan and a maintenance and traffic protection plan for the reconstruction to Interstate standards (I-78) of 13 mi of PA-309 in Lehigh County, Pennsylvania. Preconstruction uses of origin-destination surveys, traffic counts, and a public information program were undertaken. Traffic impacts were identified for each of the six construction sections. Construction methods for maximizing on-system traffic movement were identified and coordinated with section design consultants. Alternative routing and detours were identified and improvements were recommended (*14*). In addition to these traffic management techniques, GAI, PennDOT, and FHWA are also investigating construction section scheduling and the limiting of work-order changes by section in an effort to reduce travel impacts and the costs involved.

Alternative CRPEPs must allow for the wide diversity of project types and conditions that exist throughout the national highway network. The different approaches that ought to be considered in this planning and evaluation process should include the use of value engineering, quick estimation techniques, management information systems, computer-based forecasting models, and special contract administration procedures, of which several are currently being put to test in the field. The criteria for formulating alternative CRPEPs should include, among other factors,

- Levels of project complexity;
- Expected project duration;
- Estimated project costs for construction, management, inspection, and contract administration;
- Facility type (road, bridge, tunnel, number of lanes);
- Project type (reconstruction, redecking, major overlay);
- Methods and materials (prefabricated slabs, segmental versus continuous pours, polymer concretes, steel decking);
- Criticality of the link (high versus low volumes, excessive peak-period volumes, availability of alternative routes, capacity reduction required during construction);
- Estimated external costs to users, nonusers, and business;
- Direct project costs versus mitigation and external costs; and
- Totals cost versus project quality trade-offs.

SUMMARY AND CONCLUSION

The impacts of major urban transportation construction projects on existing traffic patterns and economic activity have become a major issue confronting the expedient execution of reconstruction projects. Reducing the costs of delay to the local community of users and nonusers imposed by highway and bridge reconstruction projects has become a major focus of discussion and research, as indicated by the numerous studies cited in this paper, most of which have been published within the last 5 years. That serious efforts are being taken to understand and tackle this problem at all levels of government and industry is encouraging to prospects that future reconstruction projects will be executed at far less cost to the general public than has been done in the past.

The overall condition of the highway infrastructure in most every state of the Union is seriously beyond the budget allocations available to correct it within this century. The trade-offs that exist between higher direct project costs and local travel disruptions require careful analyses so that the available funds can be expended in the most cost-effective way possible. Thus, although upward pressure should not be placed on the direct costs of reconstruction projects, strategies must be found that can be employed to successfully mitigate the external costs associated with these projects.

As described in this paper, strategically planned actions have been adopted or are being considered in the execution of several recent or planned projects to reduce construction-related impacts. Examples of such actions include (a) innovative project-scheduling strategies; (b) new construction techniques, including the use of special or prefabricated materials; (c) use of contract incentives to encourage more timely performance; (d) construction and use of temporary traffic lanes; (e) traffic improvements to alternative routes; (f) increased supply of public transportation services, (g) private and public promotion of ridesharing, and (h) public awareness campaigns via printed materials and the news media. This paper stands as an initial overview of state-of-the-art reconstruction techniques, traffic accommodation strategies, construction quality-control measures, and project development and evaluation processes as they have been applied to mitigate corridor travel impacts during reconstruction projects.

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The statements and opinions expressed in this paper are the sole responsibility of the authors.

Uses of the FREQ8PL Model To Evaluate an Exclusive Bus-High-Occupancy-Vehicle Lane on New Jersey Route 495

BERNARD ALPERN AND MARVIN C. GERSTEN

The FREQ8PL freeway simulation model was used to aid in the evaluation of the feasibility of a proposed exclusive bus-high-occupancy-vehicle priority lane treatment on New Jersey Route 495 between the New Jersey Turnpike and the Lincoln Tunnel. The input data, assumptions, and usefulness of the model in assessing impacts of alternative treatments are described. The model was used as an aid in the evaluation of three possible configurations of an exclusive lane. The simulation results indicated the importance of maximum utilization of bottleneck sections. They also indicated the importance of beginning the priority lane before the start of queues of nonpriority vehicles. The simulations revealed a significant limitation of the FREQ8PL model: it cannot account for reduced processing capability at on ramps blocked by standing mainline queues. To remedy this, an external spreadsheet-based procedure for adjusting ramp volumes was developed. This external procedure was also needed to supplement the queue length and travel time information reported by FREQ8PL to obtain estimates of queue lengths and delay times on blocked ramps. Probable shifts in route of travel in response to the priority lane implementation were also estimated external to the FREQ8PL model, because of limitations in the model's ability to estimate such shifts. A lower level-of-service F speed-flow curve than that presented in the current *Highway Capacity Manual* was developed to replicate the dense, slow-moving queues observed on this freeway during peak periods. The spreadsheet program was also used to create several useful graphics displaying projected travel times and queue lengths.

New Jersey Route 495 is a 2.5-mi-long, six-lane freeway (three lanes per direction) running in an east-west orientation between the New Jersey Turnpike and the Lincoln Tunnel (see Figure 1). With the George Washington Bridge to the north and the Holland Tunnel to the south, the Lincoln Tunnel is one of the three Hudson River vehicular crossings providing access to Manhattan. As the only expressway-type facility feeding the Lincoln Tunnel, the Route 495 mainline carries some 15,000 vehicles, including automobiles, buses, and trucks heading toward Manhattan (eastbound) in a typical morning peak period (7:00-10:00 a.m.). The capacity of Route 495, together with two local street approaches that also feed the Lincoln Tunnel, significantly exceeds the a.m. peak-period three-lane eastbound capacity of the tunnel itself (estimated at 3,900 vehicles per hour). Extensive backups occur at the tunnel during peak traffic periods.

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In December 1970, one of the westbound lanes of Route 495 was officially opened as a contraflow lane exclusively for the use of eastbound buses during weekday morning rush hours. This became the first reverse-flow exclusive bus lane in the country, allowing commuter buses to bypass automobile and truck traffic backed up from the tunnel.

During a year's testing the exclusive bus lane (XBL) carried thousands of commuters daily at a time saving varying from 10 to 25 min. In 1971 more than 206,000 buses and 8.7 million riders used the lane. Because of the favorable indications at the end of the trial year, the XBL became a permanent part of the Lincoln Tunnel operation. Since this time, the XBL has progressed in terms of increased volume and physical or operational improvements.

XBL travel time has varied as its use has increased. The free-flow travel time at a recommended speed of 30-35 mph is about 5 min or slightly less, a figure that was achieved regularly until the early 1980s. With the implementation of a nonstop toll program for buses in March 1985, average XBL travel times have been maintained in the range of 5.5 to 6 min, in spite of peak-hour bus volumes approaching the capacity of the lane.

Since this time, however, XBL use has grown rapidly and peak-hour demand has exceeded the lane's capacity. This has caused bus backups at the entrance to the lane, where bus flows from the New Jersey Turnpike and New Jersey Route 3 merge. Delays of 4 to 5 min or more regularly occur at this location during the peak hour (7:30-8:30 a.m.). As a result, alternative improvements have been discussed, including conversion of the leftmost eastbound lane, designated "Lane 3," of Route 495 to exclusive bus and carpool use.

This paper describes the use of the FREQ8PL freeway simulation model, which was selected as the most applicable existing off-the-shelf computer program, for evaluating the proposed priority bus-high-occupancy-vehicle treatment for Lane 3.

BACKGROUND ON FREQ8PL

FREQ8PL is the latest in a series of freeway simulation models developed at the Institute for Transportation Studies (ITS), University of California, Berkeley (*1*). Released in 1985, FREQ8PL was designed for the evaluation of normal-flow (as opposed to contraflow) exclusive lanes (also called "priority lanes") on urban freeways.

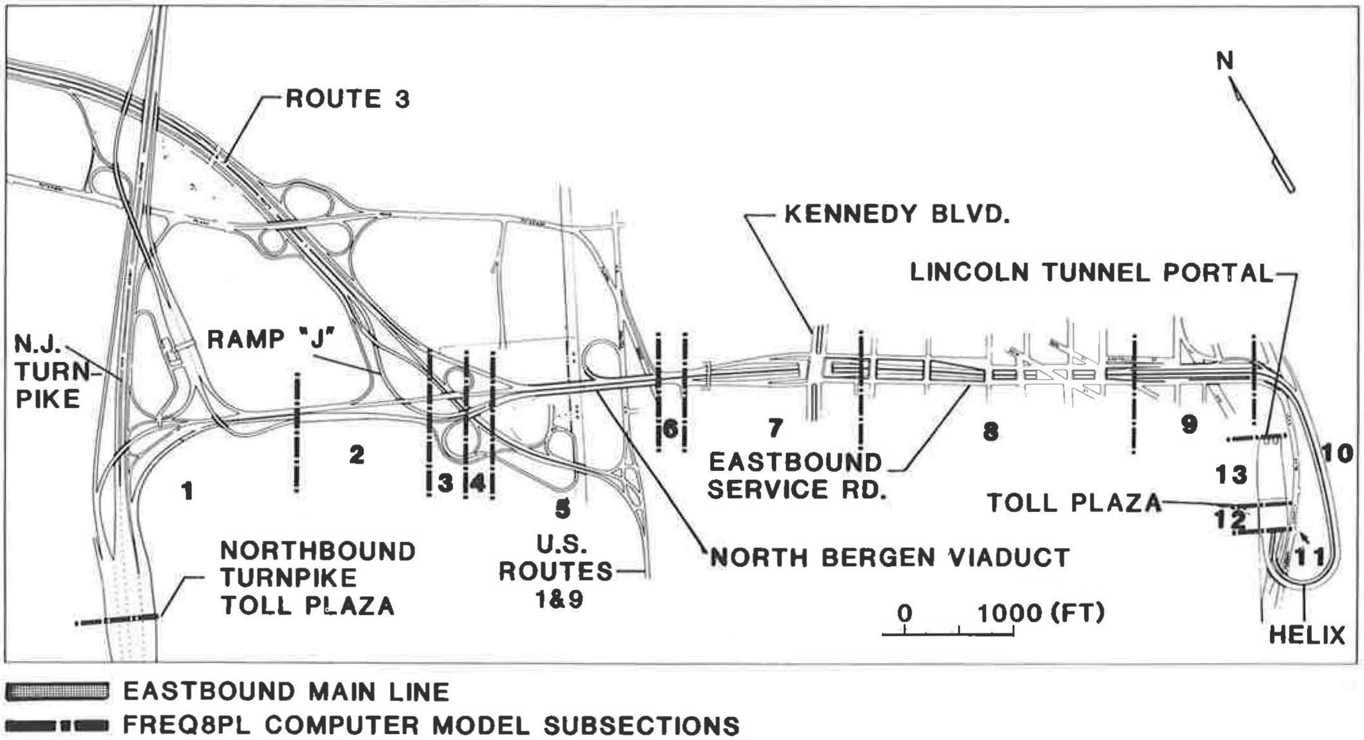

FIGURE 1 **New Jersey Route 495 showing simulation subsections.**

As described in detail earlier (*1*), FREQ8PL simulates the performance of a mainline freeway section divided into subsections reflecting major changes in demand or capacity. Inputs to the model include

- Ramp counts, reflecting the number of vehicles entering and exiting at each freeway ramp by time slice;
- Vehicle occupancy distributions (percent of vehicles by one-, two-, or three-plus-person occupancy) for each on ramp;
- A description of each subsection, including number of lanes, capacity, length, and whether the subsection has an origin (on ramp) or a destination (off ramp) or both; and
- Speed-flow curves, reflecting the relationships between speed and volume-to-capacity ratio, for each subsection type.

A submodel within FREQ8PL (called SYNPD2) estimates origin-destination trip tables by time slice (typically a 15-min period) based on the ramp counts. [A recently published article (*2*) discussed the effectiveness of using synthetic origin-destination data in freeway simulation models.] The inclusion of this submodel is one of the features that distinguishes FREQ8PL from its predecessors. FREQ8PL then performs a demand-capacity analysis for each time slice. Bottleneck locations where demand exceeds capacity are identified. The model then uses queueing theory and shock-wave theory (*3*) to calculate the extent of queueing upstream of the bottleneck locations. The speed-flow curves are used to calculate travel time in each subsection by time slice. Reports are generated showing the simulated travel times and queue locations, as well as other evaluation measures such as aggregate vehicle miles traveled, fuel consumption, and vehicle emissions.

FREQ8PL is capable of performing simulations for the following conditions:

- Before implementation of a priority lane,
- Short-term conditions after implementation of a priority lane (before route or mode shifts, or both, occur), and
- Longer-term conditions after implementation of a priority lane (after route or mode shifts, or both, occur).

FREQ8PL assumes that the priority lane is in operation during the entire time period being simulated. It also assumes that priority vehicles are free to enter and leave the exclusive lane at any point.

Additional information on the algorithms and assumptions used by the model is available in documents published by ITS (*4,5*).

For this analysis, the FREQ8PL model was installed on the Prime 550-II minicomputer located in the New York City office of URS Company, Inc.

DESCRIPTION OF MODELED STUDY SECTION

The mainline freeway section to be modeled was defined as eastbound New Jersey Route 495 beginning at the New Jersey Turnpike's eastern spur exits (16E and 17E) to the Lincoln Tunnel and continuing through the Lincoln Tunnel to New York. The modeled section includes the toll plaza for the Lincoln Tunnel, as well as the Lincoln Tunnel itself, which are operated by the Port Authority of New York and New Jersey. The Lincoln Tunnel toll plaza is made up of 14 toll lanes operated during morning peak periods, of which the leftmost two are almost entirely dedicated to the XBL and local buses.

The Lincoln Tunnel comprises three separate tubes: North, Center, and South, each carrying two lanes of traffic. The

North Tube is always westbound, the South Tube always eastbound, and the Center Tube lanes are reversed during peak periods to accommodate the peak direction flow. Thus, in the morning peak period, four tunnel lanes are available for eastbound (toward Manhattan) traffic. One of these lanes (the left lane of the Center Tube) is used almost exclusively by buses using the contraflow XBL, as well as buses entering from the local street system. The system modeled for this analysis did not include the XBL or the tunnel bus lane.

New Jersey Route 495 was divided into 16 subsections (subsections 1-13 are shown in Figure 1). A new subsection was started at each freeway entrance and exit. An additional subsection (6) was provided at the east end of the North Bergen viaduct, where it was initially assumed that the exclusive Lane 3 operation would begin.

Additional subsections were provided at the Lincoln Tunnel toll plaza, at the tunnel portal in New Jersey, at the beginning of the upgrade section in the tunnel, and at the tunnel portal in New York. Each of these represented a point where roadway capacity changes significantly.

Three possible exclusive lane configurations were tested: in the first a continuous exclusive bus-HOV lane was provided in the leftmost eastbound lane (Lane 3) of Route 495 beginning at the eastern end of the North Bergen viaduct and continuing through the Lincoln Tunnel toll plaza and into the right lane of the Center Tube (which would be entiredly dedicated to buses and HOVs). In the second configuration the exclusive Lane 3 operation ended at the Lincoln Tunnel toll plaza. In a third configuration the exclusive lane started in the left-hand lane of Route 3 (a major east-west six-lane freeway feeding Route 495), continued via the left lane of an existing left-hand ramp (Ramp J) from Route 3 to eastbound Route 495, and ended at the tunnel toll plaza.

DATA COLLECTION AND INPUT ASSUMPTIONS

Ramp classification counts and occupancy distributions were available from surveys conducted on four typical weekdays during 1984 and 1985. The period surveyed was 6:00 to 9:30 a.m. By averaging the data collected on these dates, a total eastbound demand of 13,800 cars and trucks was obtained, of which 65 percent were single-occupant passenger cars, 19 percent were two-occupant cars, 6 percent were three-or-more occupant cars, and 10 percent were trucks. Aerial photography was also used to identify times, locations, and densities of current queueing along the mainline roadway. Observations of mainline travel times at various time points throughout the peak period were also made to complete the volume-density-speed data base.

The number of lanes and length of each subsection were readily identifiable. Capacities for the freeway subsections were computed by using the conventional *Highway Capacity Manual* (HCM) [*Circular 212 (6)*] techniques. Subsection capacities were first computed in vehicles per hour by using the computed average percentage of trucks in each subsection over the 6:00-9:30 a.m. period to determine the adjustment factors for heavy vehicles. It was then decided that the wide fluctuation in truck percentages over this period made it inappropriate to use a single vehicle-per-hour figure to represent the capacity of each subsection over the entire morning peak period. Because FREQ8PL does not allow for the use of different capacities by time period for a given subsection, it was necessary to express all subsection capacities in equivalent passenger-car units (pcu). All freeway demand information was correspondingly converted from vehicles to pcu. A truck was taken to be the equivalent of two passenger cars.

Considerable care was taken in estimating the hourly capacity of the Lincoln Tunnel Toll Plaza. XBL and local buses were excluded, because it was not necessary to consider these vehicles for the simulation of existing conditions on the Route 495 main roadway approach to the toll plaza. "Audit sheets" showing the vehicles processed on November 31, 1984, at each toll lane at 14- or 16-min intervals were provided by the Port Authority. Excluding Lanes 7 and 9, in which the XBL and local buses predominate, the maximum observed processing rate for the entire toll plaza on this date was about 80 vehicles per minute, or 4,800 vehicles per hour. This value was tested as the capacity of the toll plaza in the simulation of existing conditions and was adjusted downward in order to produce simulated queue lengths that replicated observed queues and delay times as closely as possible. A final capacity estimate of 4,550 pcu/hr was obtained for the toll plaza, exclusive of Lanes 7 and 9.

The combined car and truck capacity of the three eastbound tunnel lanes during the morning peak period was estimated, using the HCM, at about 5,200 pcu/hr. This value was too high for use in the simulations, however, because the demand numbers took each truck to be the equivalent of two passenger cars, whereas in the upgrade section of the tunnel, an equivalency of 5 or 6 is more appropriate. Because the demand numbers could not be increased midstream, the tunnel capacity value had to be reduced to compensate. A capacity value of 4,400 pcu/hr for the three tunnel lanes was found to yield simulated queue lengths and delay times that were in close agreement with observed conditions.

The final hourly capacities used for each freeway subsection are given in Table 1.

TABLE 1 EASTBOUND ROUTE 495 SUBSECTION CHARACTERISTICS

Subsection No.	Length (ft)	Capacity (pcu/hr)	No. of Lanes
1	2,830	4,000	2
2	1,180	6,000	3
3	340	4,000	2
4	210	3,200	2
5	1,620	8,000	4
6	180	8,000	4
7	1,600	6,000	3
8	2,520	6,000	3
9	1,020	6,000	3
10	2,940	5,700	3
11	10	10,000	5
12	250	14,000	7
13	570	4,550	4
14	3,740	5,200	3
15	4,400	4,400	3
16	10	6,000	3

Four speed-flow curves (Figure 2) were input to FREQ8PL. Curve 1 was used to represent all sections of Route 495 from the New Jersey Turnpike to the beginning of the helix approach to the Lincoln Tunnel. Curve 2 was used for the helix and for the downgrade section in the Lincoln Tunnel. Curve 3 was used for the upgrade section of the tunnel, and Curve 4 was used for the 260 ft immedicately before the Lincoln Tunnel toll booths. The upper limits of these curves were based on speed runs performed on a Saturday morning, when traffic was very light. The remainder of each curve was adapted from the speed-flow curves in the HCM. The lower limbs of the curves (used for queued traffic, level-of-service F) are lower than those in the HCM. This results in denser, slower-moving queues, which more closely match the observed queue densities and speeds on Route 495.

SIMULATION OF EXISTING CONDITIONS AND CALIBRATION OF INPUTS

A series of simulations was performed in order to calibrate some of the key inputs to the FREQ8PL model. In particular, the capacities of three critical subsections, including the Lincoln Tunnel toll plaza and the upgrade section in the tunnel, were adjusted on the basis of the simulation outputs. The goal was to obtain capacity values that would yield simulated queue lengths and delay times in reasonable agreement with observed queues and delays.

Simulated queue lengths were compared with the queue lengths observed in aerial photographs taken on the mornings of October 16 and 17, 1985. Simulated travel times were compared with observed travel times from Port Authority runs conducted on various dates in 1985. As a result of adjusting the capacities of the critical subsections, close agreement was obtained between the simulated and observed queue lengths for specific time points during the peak period.

Simulated and observed travel times are shown in Figure 3 (produced using Lotus 1-2-3). It can be seen that the simulated times are generally in close agreement with the observed times.

SIMULATION OF SHORT-TERM CONDITIONS AFTER IMPLEMENTATION OF PRIORITY LANE

Assumptions

Short-term (or Day 1) simulations were performed for three configurations of a Lane 3 exclusive bus-HOV lane on Route 495. HOVs were defined as passenger vehicles with three or more occupants, because initial analyses indicated that a two-or-more HOV definition would overload the lane. The first configuration (called Long Lane) starts immediately east of the North Bergen viaduct and continues through the Lincoln Tunnel (with the right lane of the Center Tube being dedicated to buses and HOVs). For this configuration, it was assumed that four toll lanes at the tunnel would be dedicated to the buses and HOVs from Lane 3 and the local streets. Nine toll lanes would be available for the remaining two lanes of the Route 495 roadway and the local non-HOV traffic.

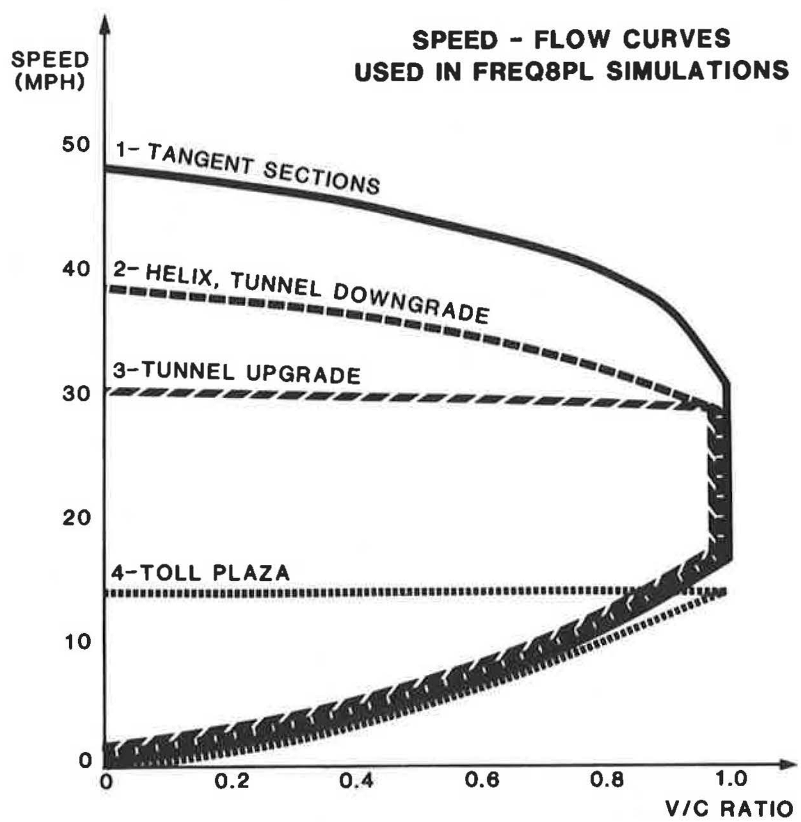

FIGURE 2 Speed-flow curves used in FREQ8PL simulations.

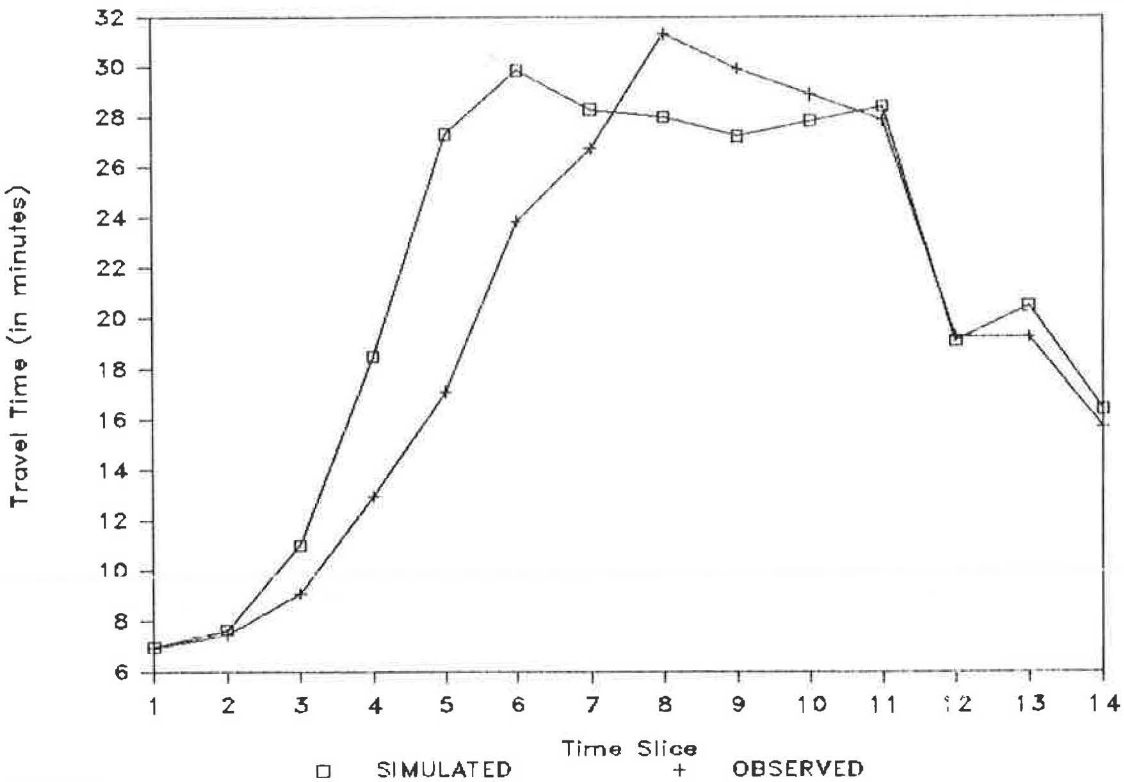

FIGURE 3 Route 495 mainline travel times: simulated versus observed.

In the second configuration (called Short Lane), the exclusive Lane 3 operation would end at the Lincoln Tunnel toll booths. Buses from Lane 3 would then be directed into either the left or right lane of the Center Tube, and HOVs would mix back in with the general traffic. The right lane of the Center Tube would be open to all traffic in order to achieve maximum use of the tunnel. For this configuration, it was assumed that only three toll lanes would be dedicated to the non-XBL buses and HOVs, whereas 10 lanes would be available to all other traffic.

A third configuration (called Short Lane 2) was based on the assumption that the exclusive lane would begin in the left-hand lane of Route 3, allowing buses and HOVs from Route 3 to bypass backups on Route 495. The lane would continue via the left-hand ramp onto eastbound Route 495 and into Lane 3, ending at the Lincoln Tunnel toll booths.

The assumed capacities of the various sections of the exclusive lane were as follows:

Tangent sections of Routes 3 and 495: 2,000 pcu/hr

Helix: 1,900 pcu/hr

Toll plaza: 1,520 pcu/hr for Long Lane; 1,140 pcu/hr for Short Lane and Short Lane 2 (380 pcu/hr/toll lane)

Downgrade section of tunnel: 1,730 pcu/hr (Long Lane only—as per HCM)

Upgrade section of tunnel: 1,470 pcu/hr (Long Lane only—one-third of 4,400 pcu/hr for three tunnel lanes)

For the short-term simulations it was assumed that no changes would occur in travel mode, route, or time period. The one exception was the assumption that some of the non-HOV traffic currently using the local street approaches to the tunnel would have to shift to the main Route 495 approach because of the need to close one of the local approaches to non-HOV traffic. This was logical insofar as this non-HOV traffic represents those vehicles currently diverting from the Route 495 mainline to the local street system for alternative routes to the Lincoln Tunnel entrances. For this analysis, these vehicles were shifted back to the Route 495 mainline by reducing the non-HOV off-ramp counts at the exits to the parallel local street.

It was assumed that the exclusive lane would be used by express buses from Route 3 (currently about 380 buses between 6:30 and 9:30 a.m. and peaking at about 190 buses from 7:30 to 8:30) as well as passenger vehicles carrying three or more occupants.

Ramp Volume Adjustments

When the short-term simulations were initially performed, a problem with the simulation algorithm was identified. The simulated queue in the non-priority lanes of Route 495 extended back beyond the northbound New Jersey Turnpike exit to the Lincoln Tunnel, blocking the other major input points to Route 495 from the southbound turnpike and from Route 3. The model assumes, however, that whatever volume is given for an on ramp is able to enter the freeway regardless of whether the entrance is blocked by a standing queue. The simulated mainline throughput is correspondingly reduced, causing the model to project unrealistically long backups.

The only way to rectify this situation was to reduce the ramp counts at the blocked ramps. Adjusted ramp volumes were calculated for each time slice during which the ramps are blocked by assuming that the maximum volume on a blocked ramp is a certain percentage of the volume on the freeway subsection into which the ramp feeds. The percentage varied by ramp depending on the configuration of the merge.

A spreadsheet-based model was constructed to calculate the adjusted ramp volumes and to keep track of the resulting queue on each ramp. When the simulation was rerun with the adjusted volumes, the mainline queue was reduced, causing some of the ramps to be blocked for a shorter period of time. This required the ramp volumes to be readjusted. This iteration was repeated several times.

Ramp queue delay times were estimated in the spreadsheet by dividing the estimated number of vehicles in the queue in each time slice by the assumed processing rate of the ramp. Ramp queue lengths were estimated by multiplying the estimated number of queued vehicles by 20 lane-ft per queued vehicle. This figure is based on the level-of-service F speed-flow curve adopted for the simulations with an assumed v/c of 0.25.

Results

The short-term simulations indicated the importance of maximum utilization of the eastbound lanes of the Lincoln Tunnel. This was demonstrated by the extent of queueing of nonpriority vehicles projected by the model. The extent of queueing on Route 495 projected by the model for the Short-Lane configuration, which achieves maximum tunnel traffic utilization, is shown in Figure 4.

Under the Long-Lane configuration, in which there are currently not enough buses and HOVs to fill the capacity of two completely dedicated tunnel lanes, the simulated queues of nonpriority vehicles grew more rapidly and extended further back along the approach roadways to Route 495.

Under the Short-Lane 2 configuration, in which the exclusive lane would begin on Route 3 itself, buses and HOVs from Route 3 would be able to completely bypass the Route 495 queue. However, the capacity of Ramp J to process non-HOVs onto Route 495 is reduced, because its left lane would be totally dedicated to buses and carpools. The spreadsheet model described earlier was used to estimate that the impact of this reduced non-HOV capacity on Route 3 would be a non-HOV backup extending up to 1.5 mi back from Route 495 onto Route 3.

In order to compare projected travel times for the various Lane 3 configurations, the origin-to-destination travel times reported by FREQ8PL had to be supplemented with the ramp queue delay times estimated by the spreadsheet procedure for the major approaches to Route 495. Projected maximum travel times from each of the major approaches to the Lincoln Tunnel's New York portal are shown before Lane 3 and for Day 1 after implementation of Lane 3 (Short-Lane 2 configuration) in Figures 5 and 6.

The FREQ8PL model computes total system passenger hours both before and after the implementation of a priority lane. However, these estimates do no include the delays that occur at on ramps that are blocked by standing traffic queues. Therefore, the ramp delay times estimated by time slice using the spreadsheet were used to supplement the mainline travel times reported by FREQ8PL in order to develop projected travel times by approach for buses, carpools, non-HOVs, and trucks. These travel times were multiplied by the 15-min volumes at each approach and again by vehicle occupancies (3.6 was used as the average occupancy of a carpool and 41.5 for a bus, based on observed conditions).

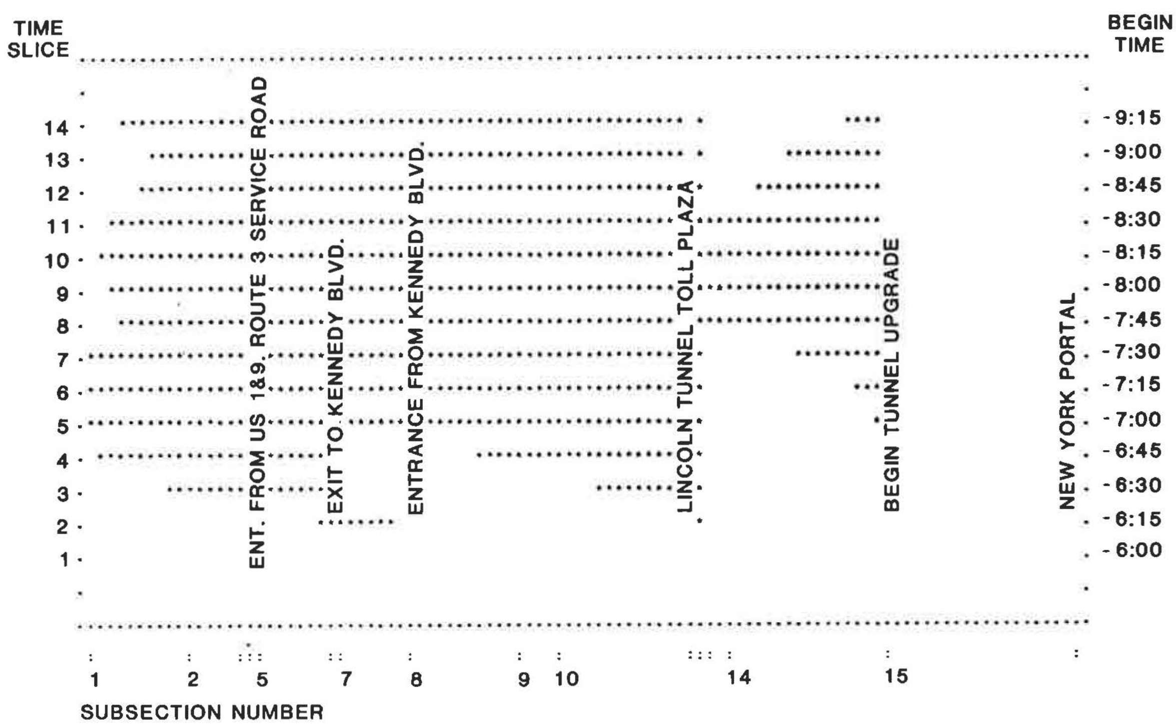

FIGURE 4 Simulation of queueing in nonpriority lanes: short-term condition after implementation of Short Lane 3 exclusive lane.

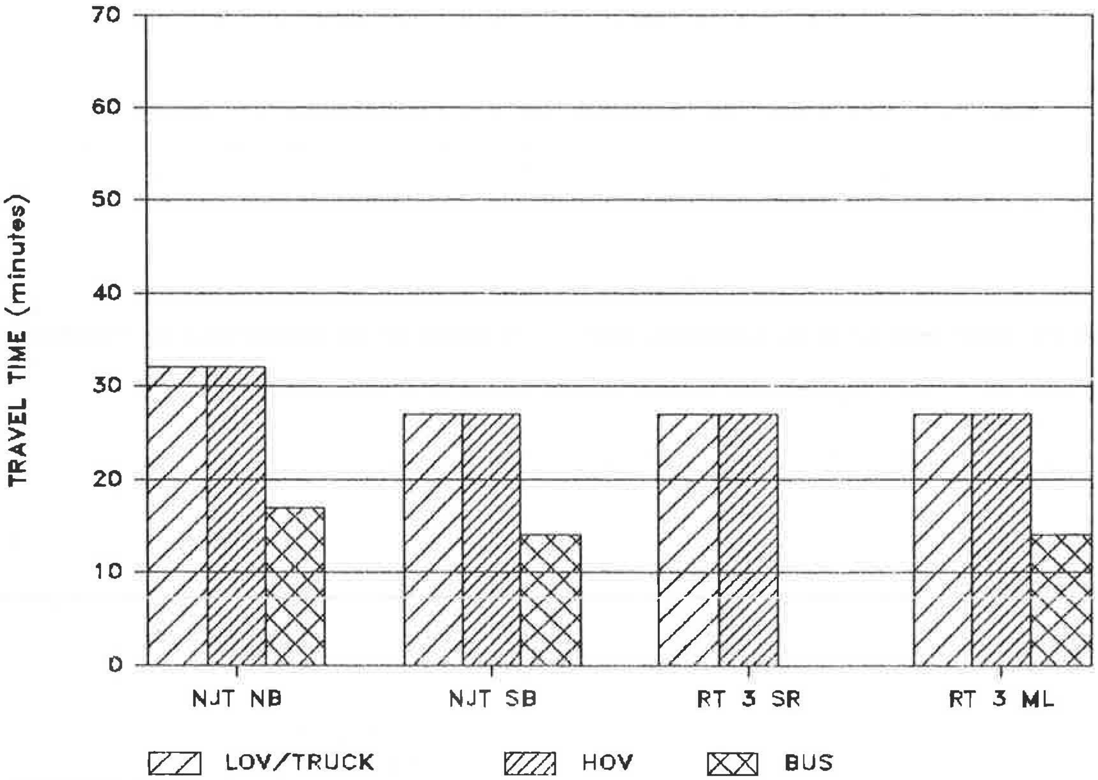

FIGURE 5 Projected maximum times to N.Y. portal before Lane 3.

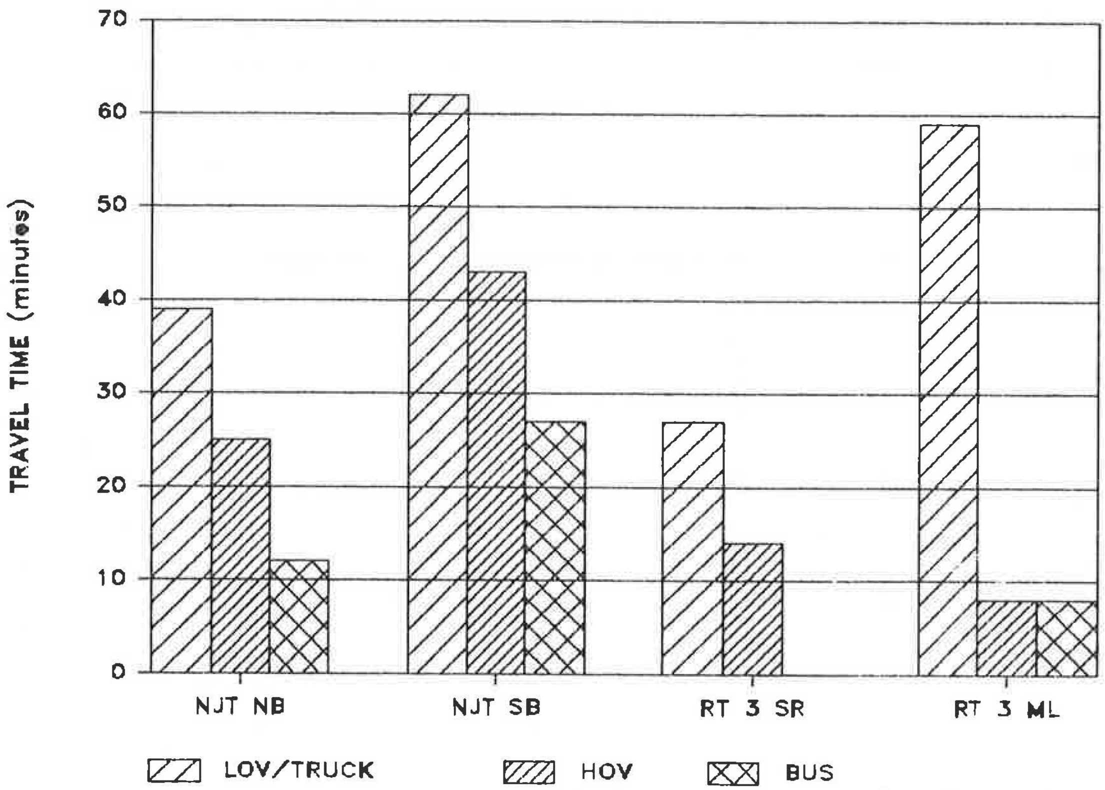

FIGURE 6 Projected maximum times to N.Y. portal, Day 1 after Lane 3.

SIMULATION OF LONGER-TERM CONDITIONS AFTER IMPLEMENTATION OF PRIORITY LANE

The Short-Lane 2 option was analyzed further to assess possible longer-term impacts after travel route shifting occurs. It was assumed that no shifts in time of travel or in travel mode would occur, even though bus and HOV travel times would be reduced relative to non-HOV travel times. At the time of the study, there was no demonstrable evidence available for the New Jersey-New York travel market, indicating that mode shifts have actually occurred in response to other HOV priority strategies that have been implemented. Accordingly, the estimation of mode shifts using theoretical models was not considered.

Assumptions as to probable diversions in route of travel were made, however. The FREQ8PL model contains a procedure for estimating diversions to a parallel alternative route. These shifts are based on a comparison of freeway mainline speeds and assumed alternative route speeds. The model does not, however, take into account on-ramp delays caused by queues blocking freeway entrances, because these delays are not calculated within the model. It was therefore necessary to estimate route diversions externally and then rerun the FREQ8PL and spreadsheet models through ramp volume adjustments to estimate the corresponding impacts on Route 495 and its approach roadways.

Two types of travel route shifts were estimated: first between the various approach routes to Route 495 and second, diversions to other Hudson River vehicular crossings.

Shifts between approach routes were estimated for each time slice by manipulating the spreadsheet model to determine the volume changes that would produce, to the extent possible, balanced travel times on the major approach roadways to Route 495.

Diversions of non-HOV vehicles to other crossings were then estimated. For lack of a more sophisticated procedure, these were calculated to produce travel times about halfway between current travel times without the priority lane and the travel times that were simulated under Day 1 conditions immediately after priority lane implementation. A total diversion of about 800 vehicles was estimated to occur during the 6:30-9:30 a.m. period under this assumption.

The model-simulated non-HOV queue lengths for each of the four priority lane conditions and major approaches (measured back from the merge point of each approach to Route 495), as estimated by the spreadsheet-based procedure, are shown in Figures 7 through 10. The lane conditions were before Lane 3, Day 1 after Lane 3, after approach-route shifts only, and after approach-route and crossing shifts.

The simulated maximum travel times from each of the major approaches after route shifts are shown in Figure 11.

CONCLUSIONS AND DIRECTIONS FOR FURTHER RESEARCH

FREQ8PL was found to be an extremely useful tool in the evaluation of the alternative priority lane treatments proposed for Route 495. As this is being written, the model is being prepared to simulate the section of Route 3 west of Route 495 to obtain more detailed information on the extent of queueing for various alternative bus and HOV priority treatments along this roadway.

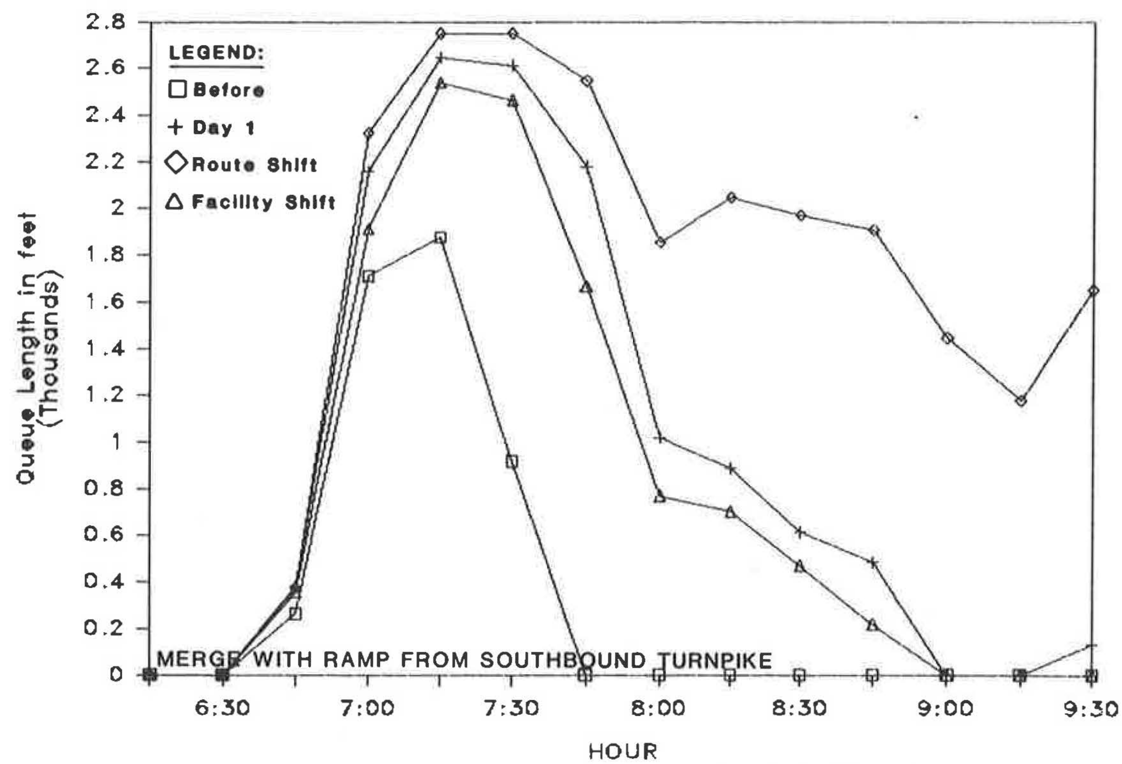

FIGURE 7 Projected queue length: northbound N.J. Turnpike approach to Route 495.

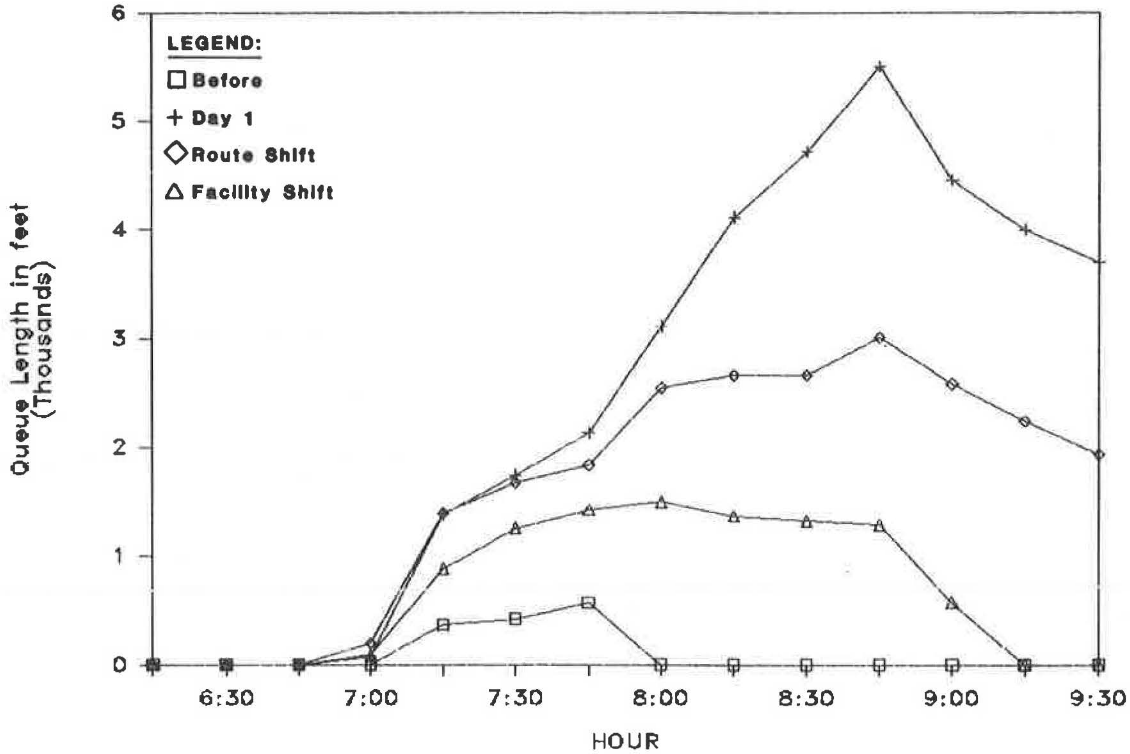

FIGURE 8 Projected queue length: southbound N.J. Turnpike approach to Route 495.

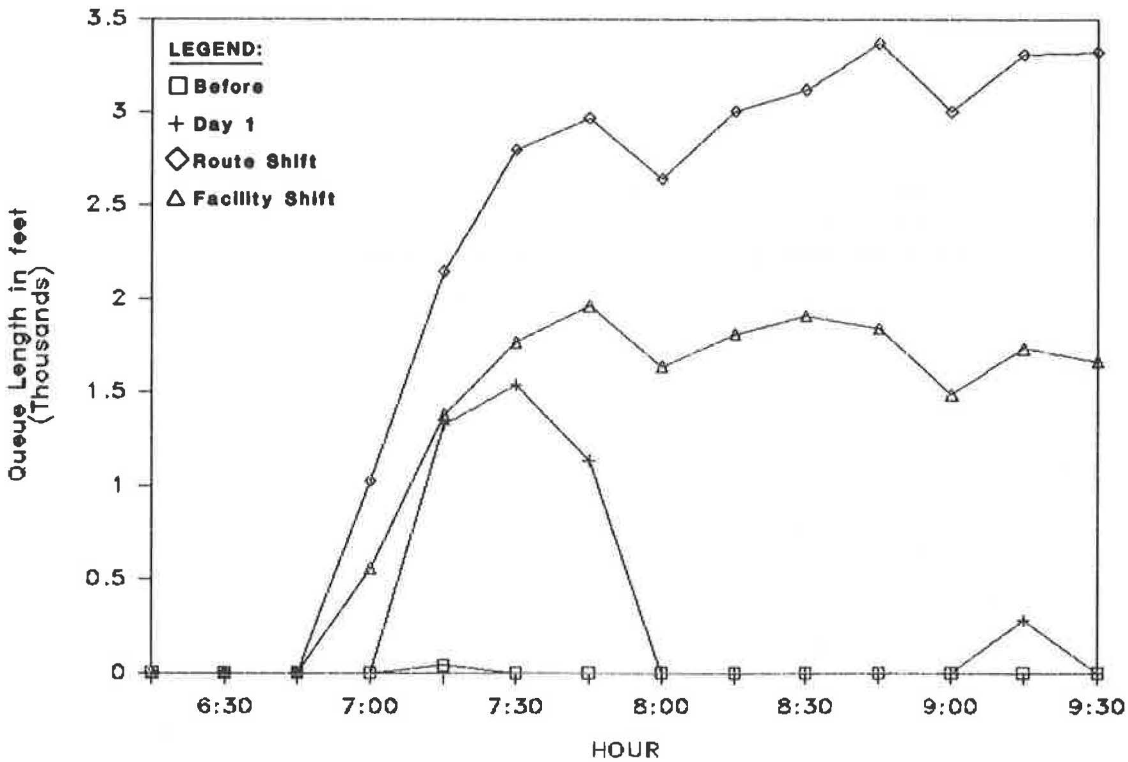

FIGURE 9 Projected queue length: Route 3 service-road approach to Route 495.

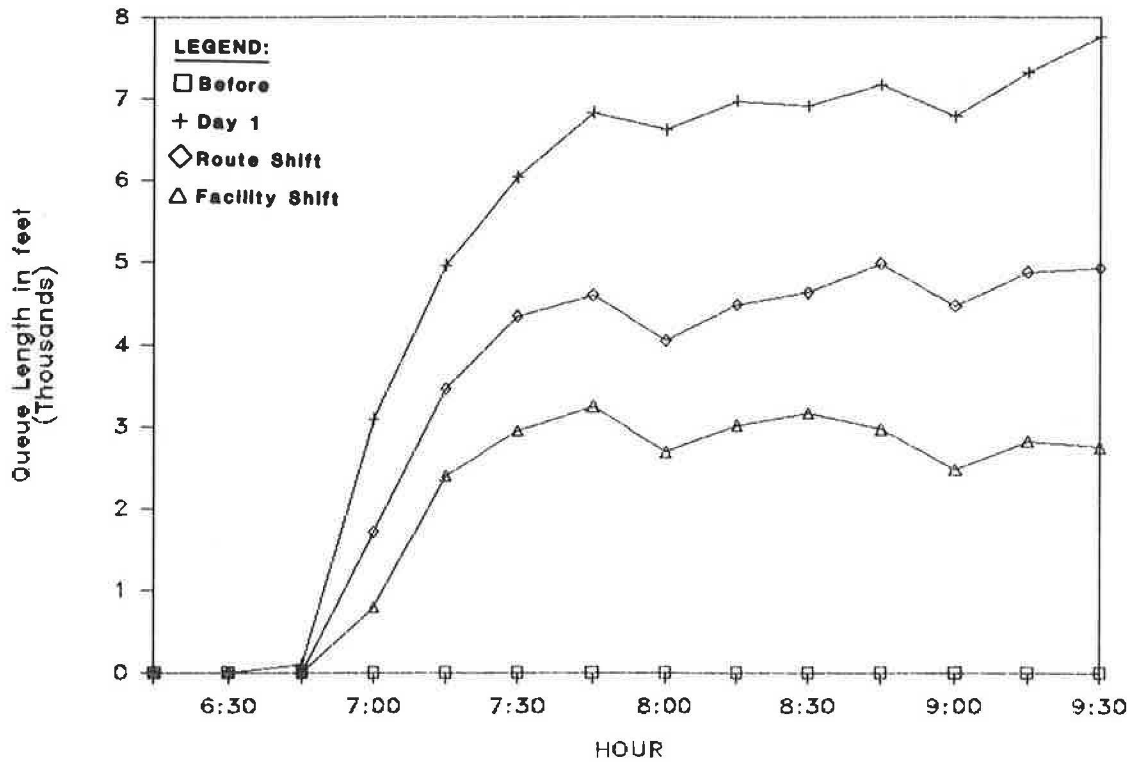

FIGURE 10 Projected queue length: Route 3 mainline approach to Route 495.

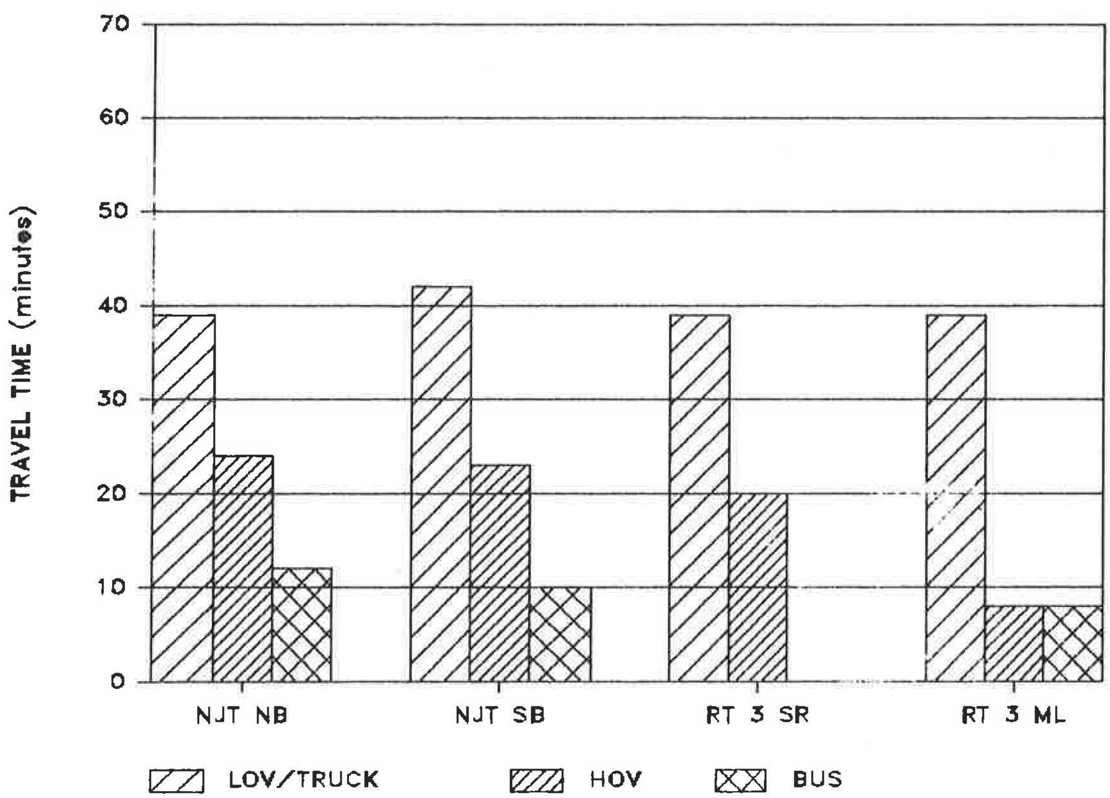

FIGURE 11 Projected maximum times to N.Y. portal: after route and facility shifts.

It should be noted that certain limitations of the model exist. In its current form, the model has no way of accounting for the reduced processing capacity of on ramps that are blocked by mainline queues. For this analysis, an external spreadsheet-based procedure for adjusting ramp volumes and estimating ramp queues was developed. This external procedure had to be relied on to supplement the queue-length and travel-time estimates reported by FREQ8PL. It is recommended that FREQ8PL itself be enhanced so that these computations can be made internally.

It would also be desirable if FREQ8PL had a more appropriate means to reflect the impact of heavy vehicles on roadway capacity. Ramp counts could be classified into automobiles, trucks, and buses, instead of only automobiles and buses as at present. Pce factors for trucks and buses could be input for each subsection. Capacities would then be expressed in passenger-car units, and the model would internally convert the demand on each subsection into these units using the equivalency factors.

Furthermore, FREQ8PL's route-shift estimation capabilities are limited, so that for a given application, route shifts have to be estimated externally.

Finally, it would be useful if FREQ8PL's reporting capabilities were enhanced to include graphic displays of travel times between specified points.

ACKNOWLEDGMENT

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A Review of Candidate Freeway-Arterial Corridor Traffic Models

M. VAN AERDE, S. YAGAR, A. UGGE, AND E. R. CASE

In order to select a model for application in Ontario's freeway-arterial corridors, a review of potential candidates was performed. The criteria for evaluating suitable alternatives included the quality of the path selection technique, the ability to represent dynamic queueing effects, the accuracy and detail of the traffic flow model, and the resolution of the traffic signal representation on parallel arterials. The following models were initially considered: MACK, FREFLO, FRECON, INTRAS, TRAFFICQ, FREQ, CORQ, CORCON, SCOT, TRAFLO, DYNEV, CONTRAM, SATURN, and MICRO-ASSIGNMENT. On the basis of a literature review and a preliminary evaluation of fundamental requirements, some of these initial models were found to be clearly incompatible with the objective of modeling dynamic assignment and queueing in freeway-arterial corridors. Of the remaining models, which included FREQ, CORQ, TRAFLO, DYNEV, CONTRAM, and SATURN, none could fully satisfy all major criteria. However, it appeared that some could potentially be upgraded, given that a considerable amount of further development effort was applied. In this respect, CONTRAM and CORQ appeared most promising because of their superior queueing-based assignment techniques and their treatment of time varying queues and demands.

The Ontario Ministry of Transportation and Communications (MTC) has an on-going need for models to evaluate traffic management schemes within a number of its freeway-dominated corridors. Specifically, models are required for application to the Queen Elizabeth Way, the Burlington Skyway, Highway 401, and the Ottawa Queensway. Within these corridors the implementation of existing routing, diversion, ramp metering, and other related traffic management strategies must be reviewed, whereas there also exists an on-going need to evaluate new candidate strategies.

At present, MTC already has numerous simulation and optimization models at its disposal for the analysis of various types of traffic facilities. However, because most of these models were developed for different purposes, they usually have characteristics that do not perfectly fit MTC's corridor-oriented needs. It was therefore not clear which of these existing models was at this time best suited for application within Ontario's freeway-arterial corridors and which of them should be considered for further development.

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The first step in the study was to develop a set of criteria for the evaluation and selection process. This was followed by a preliminary survey of the available models in terms of these criteria, which in turn resulted in a short list of models for further evaluation. The final step involved a final critique of this short list in order to arrive at recommendations regarding models to be considered for further study and development or application.

MODEL EVALUATION AND ELIMINATION AND SELECTION CRITERIA

The quality of a freeway-arterial corridor model depends not only on the presence of several different features but also on the quality of their implementation. Although some of these features are desirable but optional, others are strictly essential. However, a model's appraisal depends most heavily on the application considered, because different applications emphasize different model features and alter their relative importance. Based on these considerations, a number of evaluation criteria were developed in conjunction with MTC's traffic systems research and traffic management and engineering personnel. These model evaluation criteria guided the model review process and were classified as follows:

1. Quality of model in terms of traffic engineering theory,
2. Quality of program code,
3. User friendliness and documentation,
4. Field validation and verification, and
5. Availability, implementation, cost, and support.

Although no detailed numerical grade could be assigned to each specific criterion, the following relative rating system was found useful in assigning them relative degrees of importance:

Absolutely necessary	***
Desirable now and necessary in future	**
Desirable	*
Not important	—

A detailed listing and rating of the foregoing modeling criteria is provided in Table 1. Based on these ratings, a summary of the most important criteria (those rated ***) was prepared and checked for credibility and consistency. This summary, which guided the review and screening of the initial candidate models, is as follows:

TABLE 1 MODELING EVALUATION CRITERIA

Criterion	Importance Rating
Traffic Engineering Theory	
Freeways	
Queueing	***
Merging and weaving sections	***
Ramp metering	***
Balancing of collector and express lanes	***
Car-following and lane-changing behavior	—
Analysis of shock waves	—
Priority entry and lane provision	**
Oversaturation and queue spillback	***
Traffic signals	
Cycle length, phasing, and green split	***
Coordination and progression	**
Platoon dispersion	—
Critical intersection control	—
Oversaturation and queue spillback	***
Length of time slice effects	*
Dynamic adaptation of capacity	**
Assignment	
Queueing	***
Dynamic reassignment	***
Bidirectional corridors	**
Vehicle and facility types	*
Other factors	
Adaptive learning (day-to-day)	*
Off-line study tool	***
On-line traffic responsiveness	**
Suitability for optimization	***
Program Code, User Friendliness, and Documentation	
Program source code	
Clarity	**
Comments and internal documentation	**
Modular structure	**
Suitability for modification	***
Program efficiency and limitations	
Maximum size of network (500 links, 250 nodes)	**
Execution time (10 min mainframe, 16 hr microcomputer)	**
Efficient to run for optimization of network	**
Portability between mainframes	***
Transferability to microcomputer (PC)	**
Program Code, User Friendliness, and Documentation	
General user friendliness	
Automated input	*
Error checking and messages	*
Editing of input data	*
Synthetic data	*
Accessibility to optimization module	**
Outputs and results	
Flows, queues, travel times, speed by link	***
Graphical presentation	**
Fuel consumption	*
Emissions	*
Noise pollution	*
Summaries by classification	**
External documentation	
Description of model's theory	**
Software installation and maintenance	**
Description of model limitations	**
Interpretation of results	**
Field Validation and Verification	
Data	
Using artificial data	
Using actual data (preferred)	
User	
By model author	
By other users (preferred)	
Availability, Implementation, Cost, and Support	
Model availability	
Cost	***
Source code	***
Additional support and follow-up	***
Implementation	
Common mainframe	***
Minicomputer	**
Microcomputer	**

NOTE: Ratings are defined as follows: ***, absolutely necessary; **, desirable now and necessary in future; *, desirable; — not important.

1. Freeways
   a. Oversaturation and queue spill-back
   b. Merging and weaving
   c. Ramp metering
   d. Balancing of collector and express lanes
   e. Queueing
2. Traffic signals
   a. Cycle length, phasing, and green split
   b. Oversaturation and queue spill-back
3. Assignment
   a. Dynamic reassignment
   b. Queueing
4. Other factors
   a. Off-line study tool
   b. Suitability for optimization
5. Program source code
   a. Availability
   b. Suitability for modification

6. Outputs and results: flows, queues, travel times, speed by link
7. Model availability
   a. Cost
   b. Source code
   c. Additional support and follow-up
8. Implementation: common mainframe or minicomputer

PRELIMINARY SURVEY OF CANDIDATE MODELS

In the preliminary survey a number of different types, groups, or series of corridor-related models were considered. Some of the findings of this initial survey are summarized in the following paragraphs for the following models:

- MACK-FREFLO-FRECON series,
- INTRAS type,
- TRAFFICQ,
- FREQ series,
- CORQ-CORCON series,
- SCOT family,
- TRAFLO,
- DYNEV,
- CONTRAM,
- SATURN, and
- MICRO-ASSIGNMENT type.

As shown, not every existing freeway model was evaluated. Instead, the review concentrated on the most common types and grouped these when they had a common origin or structure, or both. In addition, some of these models are at present clearly unsuitable for modeling freeways, traffic-signalized arterials, queueing, or traffic assignment. However, because the perfect model did not exist, all imperfect models became contenders for consideration during the preliminary evaluation.

MACK-FREFLO-FRECON Series

The MACKII model (*1*) and the original MACK model (*2*) are deterministic, macroscopic models that are basically a set of conservation equations and corresponding set of speed-density equations. A later modification by Koble et al. (*3*) has unofficially been labeled MACKIII. MACK models consider incidents, but there are now no provisions for environmental impact measures or parallel routes (*4*). An evaluation was made by Derzko et al. (*5*), who found it to contain certain instabilities.

The FREFLO model (*6-8*) is a further development of the MACKII model. It contains three general control strategies, can consider incidents, and has options for fuel and emission measurements. However, it cannot model parallel routes. FREFLO is also modular and has been used by second parties (*4*). In general, FREFLO is derived from car-following theory (*2*), but its overall characteristics may also be derived from statistical considerations. Derzko et al. (*5*) also evaluated this model and found that, by virtue of an identical underlying differential equation, FREFLO exhibited the same instabilities as MACK.

FRECON (*9*) and its update FRECONII (*10*) are dynamic, macroscopic freeways simulation models developed from Payne's FREFLO model. The original version simulates freeway performance and generates point detector information for calibration and validation. The model can interact with control programs in order to evaluate pretimed, local traffic-responsive, and segmentwide control strategies. Incident simulation is also possible. Traffic data must be included in the form of on- and off-ramp volumes and volumes of mainline traffic. Optional inputs pertain to detector location and incident description, and the outputs include contour maps of traffic performance measures and time profiles (*11*).

FRECONII contains enhancements to simulate alternative routes (surface streets), as in a corridor. It can simulate a freeway with mixed modes of ramp metering, and the driver's spatial diversion due to ramp metering. Additional outputs include surface street performance, corridor performance, and effects due to occupancy and diversion (*11*).

INTRAS Type

The INTRAS (INTegrated TRaffic Simulation) model (*12*, pp. 95-107; *13*) uses network theory to interrelate freeway and arterial traffic. It is a stochastic, microscopic model especially developed for studying freeway incidents. Its basis is a vehicle-specific time-stepping simulation designed to represent traffic and traffic control in a freeway and surrounding surface street environment (*14*, pp. 23-32).

The program is quite large and complex in order to model all vehicle movements in the corridor. A few control strategies are incorporated into the model, but it may be difficult to allow for access of new control strategies because of the model's structure. Traffic detectors and fuel and emission data are simulated directly from the microscopic flow (*4*). Users of INTRAS have reported problems with some aspects of traffic behavior (*15*), such as vehicles that merge from acceleration lanes, vehicles at exit ramps, and the method of assigning destinations. Some of these problems relate to the complications of communication between vehicles across link boundaries.

FOMIS (*15*) provides a revised model structure that is intended to streamline the simulation process by restricting it to the freeway only, eliminating the link structure and reducing vehicle processing to a single scan. Full derivation of the car-following and lane-changing algorithms is given by Bullen (*16*) and by Bullen and Cohen (*17*). The model is said to be primarily intended as a supplemental tool to current macroanalysis methods.

TRAFFICQ

TRAFFICQ is a simulation model of pedestrian delay, vehicle queueing, and platooning behavior. It takes into account dynamic and stochastic variations, varying road widths, and movements temporarily blocked by other vehicles. Complex control techniques such as linked signals or vehicle- or pedestrian-actuated signals may be modeled, as may priority junctions. Each vehicle or pedestrian is modeled as an individual entity, and the output gives distributions of queue lengths, travel times, and pedestrian delay (*18*, pp. 161-183; *19*).

TRAFFICQ is both dynamic and stochastic. It models, for example, both varying flow levels and random variations in discharge rates of vehicles from stop lines. The technique is aimed at relatively small-scale systems or sometimes just complex isolated junctions.

The program is written in ICL's CSL simulation language, which moves vehicles in discrete time increments (4 to 6 sec). The model is divided into a series of "activities," which are scanned sequentially, and the instructions within them are only performed if a particular condition is met. Such use of simulation permits tracing of dynamic conditions and evaluation of consequences of short-lived effects. In addition, it permits the evaluation of stochastic factors through use of a frequency distribution for the derivation of some traffic parameters. Routes taken by vehicles are prespecified by the user. This makes multirouting possible, but also implies that no internal assignment technique is present. Because each vehicle and pedestrian is considered an individual entity, temporary blockages and queue spill-backs can be modeled in detail.

FREQ Series

Since 1968 the FREQ family of freeway models has been under continuous development at the University of California (*20*). These models are macroscopic and are intended to evaluate a directional freeway and its ramps on the basis of ramp origin-destination (O-D) information. Some diversion to parallel alternatives is considered for vehicles queued at on ramps, but this treatment is not very detailed. Specialized versions of the general model are available for the evaluation of lanes on freeways reserved for carpools or buses, or both, and of priority and normal entry control.

The major input to most FREQ models is a set of O-D tables for each interval or time slice (typically about 15 min). These tables would correspond to volumes or percentages of various vehicle-occupancy classes. The model can calculate the effect of weaving on capacity, and speed-flow relationships can be selected or specified by the user. Ramp characteristics must be input. The model adjusts supply and demand, and predicts a time stream of impacts that includes both spatial and modal traveler responses (*21*). The output consists of freeway performance tables containing travel time, speed, ramp delays and queues, fuel consumption, and emissions.

FREQ6PL (*22*) is used primarily for the evaluation of a freeway lane or lanes reserved for carpools or buses, or both, and FREQ7PE (*23*) was developed primarily for the evaluation of priority and normal entry control on a directional freeway. The latter program simulates the system, optimizes a control strategy through linear programming, and predicts traffic performance and traveler demand responses. Also produced are metering plans, contour maps, and impacts of priority-lane operation (*11*).

CORQ-CORCON Series

CORQ (CORridor Queueing) (*24-26*) is a dynamic assignment technique for allocating time-varying O-D demands to a time-dependent traffic network. The technique models the impact of queueing and ramp metering on traffic assignment within a freeway-arterial corridor. CORCON (*27*) is a modification of the original CORQ program but contains essentially the same core model logic. Consequently, it is not treated separately in further discussions.

CORQ considers time-slice O-D movements for a freeway-arterial corridor and assigns these in accordance with separate minimum-path and equilibrium considerations for each time slice. Traffic flows that are unable to reach their destination within the given time period because of capacity constraints are queued and carried over for reassignment to the network during the subsequent time slice. Vehicles are assigned in variable-sized increments, depending on the capacity of the links of the network, until the entire O-D matrix for a given time slice has been assigned. The solutions for each time slice are then iterated until equilibrium is reached before the analysis proceeds to the next time slice.

Traffic flows are approximated as fluids, and travel times are calculated as simple step functions for both free-flowing and congested (queueing) conditions. The model considers primarily a directional freeway, its ramps, major cross streets, and any competing alternative surface streets. Turning movements can be accounted for, but no explicit modeling of traffic lights or any progression effects takes place. These effects must be input indirectly as link characteristics.

SCOT Family

SCOT (Simulation of COrridor Traffic) (*28, 29*) is the synthesis of two previous models: UTCS-1 (Urban Traffic Control System-1) (*30*) and the DAFT (Dynamic Analysis of Freeway Traffic) model by Lieberman (*31*), with later modifications (*32*).

UTCS-1 is a microscopic simulation of urban traffic, in which each vehicle is treated as an individual entity as it traverses its path through a network of urban streets. Routing is performed on the basis of specification of turning movements. DAFT is a macroscopic simulation model of freeways, ramps, and arterials. Vehicles are grouped into platoons and lose their individual identities. Platoons are moved along the freeway according to a single prespecified speed-density relation. On nonfreeway links, they travel at the specified free-flow speed for each link and are delayed at traffic signals on the basis of their g/c ratio and the amount of traffic.

For each entry link at the periphery of the study network, traffic volumes are specified according to their destination node. The model distributes the resulting platoons of vehicles over the network according to minimum-cost paths, which are calculated frequently on the basis of current conditions.

Whenever a platoon reaches a network node, its turning movement is dictated by its minimum-cost path as it exists at that instant of time. Hence the model produces a dynamic assignment of traffic as a by-product of the simulation. Although ramp metering is allowed, the inclusion of new control strategies is restricted by the difficulty of program modifications due to the model's structure (*4*).

TRAFLO Type

TRAFLO (*33*) is a system of four traffic simulation models and one traffic assignment model. Essentially, the assignment model calculates the flows on each link, which are subsequently evaluated by using one or more of the simulation models.

Traffic assignment is performed with the TRAFFIC model (*34, 35*), which requires use of the Bureau of Public Roads (BPR) link travel time relationship. By using a representation of Wardrop's first principle (*36*), TRAFFIC assigns a specified trip table to a network that is compatible with the four simulation models. One or more of the simulation models are then used to describe traffic operations in each subnetwork at the desired degree of detail. The user may partition the analysis network into several subnetworks if more than one simulation model is to be used concurrently, but in that case interface nodes must be specified at the junctures.

The following is a brief description of the four component submodels:

1. Urban Level I Model (NETFLO I) is the most detailed; each vehicle is treated as a separate identifiable entity and three vehicle-type distinctions are permitted (automobiles, trucks, and buses). The simulation moves vehicles on the basis of activation times and leaves them dormant between activation times.
2. Urban Level II Model (NETFLO II) is supposed to be an extension and refinement of TRANSYT because the traffic stream is represented in the form of movement-specific statistical histograms. The simulation uses five histograms: Entry, In, Service, Queue, and Out.
3. Urban Level III Model (NETFLO III) is used for the network's major arterials: collectors, distributors, circulators, and connectors. These routes connect traffic generators or high-density areas.
4. The Freeway Model (FREFLO) is said to be an extension and refinement of the MACK model developed by Payne et al. (*37*). Traffic is represented through a fluid-flow analogy considering measures such as flow rate, density, and speed.

DYNEV

DYNEV was developed to estimate evacuation travel times in Emergency Planning Zones (EPZs) as part of the larger software system developed for the Emergency Exercise Simulation Facility (*38*). Its main components are an interactive input routine called PREDYN and a software system called I-DYNEV.

DYNEV is essentially an iterative procedure starting with a data input routine and followed by an assignment procedure and the I-DYNEV traffic simulation model. The simulation model computes network performance measures based on the traffic volumes and turning movements generated within the assignment. Further intermediate steps are possible to modify any controls or the trip table, or both, but then additional model iterations are required. The final analysis is complete when the output of the simulation model is compatible with the assumptions on which the original assignment is based.

The assignment model identifies the best travel times for people to move from specified origins within the EPZs to destinations just outside. It uses a modified TRAFFIC algorithm, but travel times must be calculated based on the BPR relationship of travel time versus volume. The traffic simulation model takes as inputs link volumes and turning movements from the assignment model and replicates the dynamic (time-varying) movements of the traffic stream on all roadway sections. The model is an adaptation of TRAFLO Level II in which the traffic stream is described in terms of a set of link-specific statistical flow histograms. Both the assignment and the simulation model interact with the traffic capacity submodel, which computes service rates by turn movement.

CONTRAM

CONTRAM (CONtinuous TRaffic Assignment Model) is a traffic assignment and evaluation package that models traffic flows in urban networks consisting primarily of signalized, priority, and give-way junctions (*39, 40*). However, at this time there are no freeway (motorway) modeling provisions.

Traffic demands are expressed as O-D rates for each given time interval. These O-Ds are converted into an equivalent number of vehicle packages, which are assigned to the network at a uniform rate for each time interval. Each such packet is indivisible and travels along its own individual minimum path to its destination. For each link along its path, flows and travel times are updated, whereas for each vehicle packet a record is kept of the links used and the arrival time at that link. With the latter information, each vehicle packet can be conveniently removed from the network during any subsequent iterations and a detailed queue diagram can be constructed for each link. A traffic assignment equilibrium is achieved through iterations in which each vehicle packet in turn is removed from the network and reassigned to its new minimum path. Such reassignments consider each driver as truly a marginal user and continue until virtually all reassignments result in the same paths.

The total link travel times are calculated on the basis of any oversaturation delay due to extended queueing, the duration of the red indication at traffic signals, and any random delay effects due to randomness in either arrival or departure rates. As traffic volume estimates become available from an initial assignment, delay functions for traffic signals can be updated to reflect optimized signal splits or cycle lengths, or both, but coordination between adjacent signals is not considered.

SATURN

SATURN (*41, 42*) is a traffic assignment model based on a detailed simulation of intersection delays and an assignment that employs a more general travel time relationship that is derived from the detailed simulation.

Intersection delays are determined primarily by using cyclical profiles, in a fashion much like that used in TRANSYT. Consequently the effects on delay of coordination of signal timings and platoon progression can be accounted for. On the basis of delay estimates at free-flow conditions, at the conditions modeled using the cyclic profiles, and at capacity, an aggregate power curve is fitted to represent delays at any approach volume. This power function is further supplemented with a queueing relationship for oversaturated conditions.

Traffic flows on each network link are estimated by using a weighted combination of all-or-nothing assignments. These new estimates of link flow are then reevaluated with the cyclic profile approach until equilibrium is reached between the evaluation and the assignment. During each iteration, changes in delay due to shifts in the magnitude and structure of vehicle platoons can be included and any impact of changes in opposing-direction flows can be reflected.

SATURN models two types of queues, namely, transient and permanent queues. The account of transient queues, which build up every cycle during the red phase, permits signal coordination to be evaluated but not optimized. The account of permanent queues, which develop when queues exist during the entire cycle, considers the impact on increased travel time directly and the impact on downstream links indirectly.

MICRO-ASSIGNMENT Type

MICRO-ASSIGNMENT is a microscopic adaptation of traditional transport planning assignment techniques. Traffic is assigned in a conventional fashion, but the network is coded in considerably more detail, so that individual movements or lanes, or both, can be considered (*43, 44*).

The network is coded by using an "off-set" system of network representation, in which the nodes are located along the approaches to an intersection, and each permissible traffic movement at the intersection is represented by a separate link.

Two types of delay are considered: zero-volume delay and congestion delay. The former is delay in the absence of other vehicles (acceleration, deceleration for turns, and Stop and Yield signs). The latter is delay due to traffic interference by other vehicles (queueing at signals or caused by conflicting traffic). Originally these delay relationships were based on theoretical formulas, but currently an empirical basis is used.

Assignment is based on an iterative multipath procedure that deals in time periods from 6 min to 24 hr. The technique assigns time-slice O-D patterns to the links in the network so that arrival rates and updated delays can be derived. Although queueing conditions are not modeled explicitly, the higher delays associated with oversaturation are considered in the assignment.

ELIMINATION OF MODELS FROM PRELIMINARY SURVEY

A comparison of the basic features of the initial candidate models is provided in Table 2, which also traces the models' roots and indicates their type and design purpose. To assist in the inclusion and elimination process, any further standout characteristics are noted under the headings Critique and Desirable Features.

A survey of Table 2 indicates that no one model is comprehensive in being able to model all the required factors at the desired level of detail. However, it is clear that some models are more suitable for further development and others clearly are not. Consequently some models were further examined in greater detail, whereas others were eliminated from further consideration, as will be discussed. It should be noted, however, that the elimination of certain models on the basis of freeway-arterial corridor criteria does not imply that they could not be effectively used for other applications that more closely match the models' capabilities or design objectives.

Models Not Suitable for Further Study

Because of the importance of assignment and reassignment in freeway corridors and networks, MACK, FREFLO, FRECON, INTRAS, and TRAFFICQ were eliminated. Each model is very precise in its treatment of traffic flow details, which is important when the dynamics of single facilities are studied. However, the lack of a true assignment procedure often causes these models to analyze in excessive detail traffic flows that, because of traffic diversion and reassignment, are not necessarily correct. Although some models do consider diversion, simple diversion is inadequate when several significant arterial alternatives exist within a corridor.

Alternatively, MICRO-ASSIGNMENT contains a network traffic assignment technique. However, this technique does not consider the details of the dynamics of queueing, which are at the root of the corridor problem. Although delays resulting from oversaturation during a given time slice can easily be accounted for, the impact of the resulting queues on subsequent time slices cannot be considered. The assignment technique employed in this program is in essence a very detailed version of traditional transport planning approaches, which is difficult to modify to accommodate the needs of oversaturation or queueing.

Although the SCOT model appears to satisfy most of the primary criteria, the model is no longer supported. Furthermore, the same authors have subsequently developed TRAFLO and DYNEV, which are said to be improvements over SCOT. Consequently, SCOT was dropped from further consideration.

TABLE 2 INITIAL CANDIDATE MODELS

	Assignment		Traffic Flows						
Model	Technique	Other	Freeway	Signals	Roots	Model Type	Model Purpose	Critique	Desirable Features
FREFLO	---		Conservation equation, dynamic speed-density (fluid flow)	---	MACK	Deterministic, macroscopic	Freeway (one direction)		
FRECON	Diversion due to ramp metering		Modified from FREFLO	Simple travel time	FREFLO	Macroscopic	Freeway (one direction)	Diversion not same as assignment	Adaptive discretization of step size
INTRAS (FOMIS)	---		Vehicle-specific, time-stepping simulation	---	NETSIM	Stochastic, microscopic	Study freeway incidents	No O-Ds	
TRAFFICQ	---	Paths specified by user	---	Individual: vehicles, pedestrians	Original	Microscopic	Urban network	No O-Ds	Considers pedestrians
FREQ	Diversion of ramp queue	Considers only subgroup for reselection	HCM (speed-volume)	Simple travel time	Original	Macroscopic	Freeway + evaluate: priority lanes and priority entry	Diversion not same as assignment	Linear programming optimization
CORQ	Incremental/iterative	Reassignment of queued vehicles	Step function, travel time	Implicit in travel time calibration	Original	Macroscopic	Queueing in freeway corridor	Not user-friendly, directional assignment	Automatic network performance plots
CORCON	Incremental iterative + traffic diversion		Step function, travel time	Implicit in travel time calibration	CORQ	Macroscopic	Queueing in freeway corridor	Problems with implementation	Fuel consumption and emissions
SCOT	UTCS-1: turning movements; DAFT: minimum path		DAFT, platoon flow based on speed-density	UTCS-1: individual vehicles	DAFT, UTCS-1	Microscopic and macroscopic	Test real-time control policies for corridors	Model component incompatibilities; model no longer supported	Composite network
TRAFLO	TRAFFIC *(34)*	Planning oriented, nonqueueing	FREFLO	I: NETSIM, II: TRANSYT, III: WEBSTER	FREFLO, NETSIM, TRANSYT	Microscopic and macroscopic	All networks	146,000 statements; reassignment of queues? Traffic—no queue considerations	Composite network
DYNEV	TRAFFIC *(34)*	Planning oriented, nonqueueing	Flow histogram (NETFLO II)	Flow histogram (NETFLO II)	TRAFLO	Mesoscopic	Emergency evacuation	Nonqueueing assignment	Detailed model of approach lane section
CONTRAM	Incremental/iterative (packets of about 10 vehicles)	Reiterates over entire peak (traces paths)	---	Delay = $f(G/C, C, V/C)$ + queue delay platoons	Original	Mesoscopic	Evaluate urban signalized and unsignalized network	Modeling of freeways	Good assignment; recalculation of signal timings
SATURN	All or nothing		---	Cyclic file: $TT_n = Ao + BX^n + QT$	Original	Mesoscopic	Evaluate urban signalized and unsignalized network	No freeways; very coarse assignment of queues	Considers progression between signals
MICRO-ASSIGNMENT					Traditional transport planning	Macroscopic		No queue reassignment	Simple structure

Models Retained for Further Study

Candidate models were retained for further study if they appeared to contain the required components or incorporated a model structure that was sufficiently flexible to be amenable to the required modifications. On the basis of these criteria, the following models were retained:

- FREQ
- CORQ
- TRAFLO
- DYNEV
- CONTRAM
- SATURN

The first two, FREQ and CORQ, are the most traditional types of freeway-arterial corridor models, because they consider primarily the freeway and any important or relevant parallel arterials. The FREQ series was retained for further analysis because it is a virtual standard for freeway models, and because it has a number of features not available in other models. The CORQ series was included because it appears to be the only model type that simultaneously considers queueing and reassignment in a freeway-arterial corridor.

The next two models, TRAFLO and DYNEV, have a more network-oriented structure; they can consider assignment in a multidirectional network and appear to model in detail the different facility components. They were retained because the possible increased level of detail could permit very accurate modeling, whereas their networkwide approach is rather unique.

The final two models, CONTRAM and SATURN, are primarily traffic signal-oriented assignment models. They do not currently contain any freeway logic, but because the important capability of modeling traffic assignment in a network that includes traffic signals, they were retained for further study. Furthermore, their traffic modeling and assignment approaches appear to be suitable for extension to include freeways as another link type.

DETAILED MODEL CRITIQUES

The models retained for further study were examined in greater detail in the second phase of the study. Detailed model descriptions prepared as part of this second phase have been provided by Van Aerde and Yagar (*45*), and because of their length are not repeated here. Instead, this section concentrates on the model critiques, which are based on and derived from these detailed descriptions.

The critiques are negative at times, because they often concentrate on limitations, rather than emphasize strengths. However, the focus on limitations is a necessity, considering the objectives of the study. The authors apologize to any authors whose model documentation and descriptions may have unintentionally been misinterpreted. Every effort was made to have reasonable safeguards against this, and a number of the authors were actually consulted directly. Any comments on these critiques would be appreciated.

Finally, because the authors have some vested interests in the CORQ model, every attempt was made to remain unbiased in the evaluations. It is possible that they have been more critical of CORQ than of any other model because of their knowledge of its potential shortcomings.

FREQ

The strength of FREQ models lies in their diversity of traffic impact measures, the comprehensive range of responses that are included, and the extensive field testing that has taken place. Their primary weakness is in relation to the approximate terms in which parallel alternatives are modeled, especially because of the lack of full assignment technique.

Modeling the flow of traffic on any parallel alternatives in only approximate terms may result in potentially large errors when several significant alternatives exist, especially because shifts in path selection decisions are often based only on small changes in relative travel times between competing alternatives. Furthermore, the diversion technique considers only path reselection of those queued on freeway on ramps. It ignores any path reselection from the freeway to the surface street (when the freeway is busy but not congested) and any path reselection from arterial routes to the freeway (if freeway performance improves significantly). All these limitations restrict the use of FREQ to the analysis of only the freeway or a very narrow freeway-based corridor.

Other approximations are made when queues are modeled without taking spillback into account. This affects estimates of downstream volume and the blocking of upstream traffic. In addition, because the destination pattern of queued vehicles is not retained for use in subsequent time slices, considerable errors in downstream traffic volume estimates may occur if O-D patterns change significantly between time slices.

A feature unique to FREQ is its use of a linear program to optimize ramp-metering rates.

CORQ

The main strengths of CORQ lie in its ability to incorporate the effects of dynamic queues into an assignment methodology that uses corridorwide time-slice O-D demands. Its primary weakness is in the lack of an evolutionary sequence of revisions during development and application to case studies. As a result, the actual code is ill-formatted, the output is not very user-friendly, and some obvious simple refinements to the technique are missing.

Of greatest practical significance are the resolution and generation of the current travel time relationship, which is expressed as a static step function. Especially on parallel arterials, the insensitivity of this relationship to changes in signal timings, such as cycle length and green-time allocation, is a major drawback.

Further limitations of the current program are its specialization to unidirectional travel and limitations on the trip length for which the assignment's treatment of nonqueued vehicles is valid. These restrictions are significant when there is a significant opposing-direction flow or when a large percentage of trips are longer than one time slice.

A theoretically less significant problem relates to CORQ's current output format, which tends to provide only raw simulation results. These require significant amounts of further hand processing. Instead, concise summaries and graphics of the relevant flows, travel times, and queues would make it significantly easier to use the model. In addition, the model's use and operation are not very well documented.

TRAFLO

TRAFLO is a combination of a variety of related traffic simulation and assignment models. However, incompatibilities between these component models result in certain limitations for the entire package, especially when it is used for freeway-arterial corridor applications.

TRAFFIC is a good assignment model for planning when behavior according to the BPR equation is valid. However, in terms of traffic operations, the inability of TRAFFIC to deal with queueing, non-steady-state traffic conditions, and dynamic assignment is detrimental. Specifically, the author of TRAFFIC indicated that the model should not be used for networks in which demand exceeds capacity, because the model assumes that link demands in excess of capacity will still be served. Consequently, downstream links are modeled as being loaded with larger-than-actual traffic demands, spillback from these links onto upstream links is ignored, and downstream demand is underestimated when any accumulated queues are served in subsequent time periods.

The availability of different modeling approaches permits the user to tailor the level of detail and accuracy to the specific needs of the various parts of the network. However, each of these models has different operating procedures and assumptions, leaving the user with a mixture of different network performance measures. This makes evaluation difficult and may render any global optimization virtually infeasible. Finally, the current TRAFLO model structure does not contain a feedback loop from the evaluation model to the assignment model. Consequently, the TRAFLO assignment is performed by using a highly simplified relationship of travel time versus traffic flow, resulting in a detailed evaluation of potentially very poor traffic flow estimates.

DYNEV

DYNEV and TRAFLO are very similar in terms of their authors, model philosophy, and many of the model routines. DYNEV does provide some significant improvements over TRAFLO, but the adaptation of the same core structure appears to have left DYNEV with the same fundamental limitations in terms of freeway-arterial corridor applications. Furthermore, DYNEV's use is further limited because its code is currently classified as being proprietary.

The most significant improvement in DYNEV is the introduction of a capacity submodel that permits capacity estimates to be updated within the assignment procedure. This should improve the overall accuracy of the assignment, because the results of the assignment and the evaluation will be more consistent. However, this modification does not correct for any of the queueing problems that were previously also outlined for the assignment procedure present in TRAFLO.

In contrast to TRAFLO's four evaluation submodels, DYNEV models all links using only NETFLO II (a method that models traffic flows as a series of flow histograms). Although one may question the compatibility of TRAFLO's four evaluation submodels, there are also some concerns about using a statistical histogram to replace the functions previously performed by NETFLO I (NETSIM) and FREFLO. Further difficulties derive from the need with DYNEV (and TRAFLO) to model a peak period of, say, twelve 15-min time slices (3 hr) as essentially 12 independent runs, one for each time slice. This is inefficient in terms of the person who must use the model but, more important, it appears to limit the representation of any significant interaction between consecutive time slices and the queueing that links them.

CONTRAM

Although CONTRAM is currently limited to traffic signal applications rather than freeways, its queueing-based dynamic assignment technique makes its model structure superior to that of most current models.

In contrast to traditional models, which generally consider only the last demand increment as being truly "incremental" users, CONTRAM permits each vehicle packet in turn to be a marginal user. As a result, each network user decides on his path seeing a fully loaded network rather than a network that has only been loaded to the extent of the previous increments. A second feature of this assignment technique is that all vehicles passing through a queued link are queued for a short time (depending on the current queue size). By contrast, in most other models a quantity of vehicles equal to the link's saturation flow is not queued at all, and all additional vehicles are queued for a full time slice. Finally, the assignment technique circumvents the approximation of most previous models, which required all vehicles to reach their destination within one time slice unless caught in a queue. The CONTRAM approach permits vehicles to take more than one time slice, even if they are not queued at any point along their path.

CONTRAM's main shortcoming in terms of this study is its lack of model routines for freeways and freeway merging and weaving sections. Such an addition is certainly not a trivial task, but there appears to be no major obstacle within the model's structure to prevent such an enhancement. Of further concern is CONTRAM's extensive use of memory and computer time, which may be important when much larger corridors or networks are considered. Finally, unlike SATURN, CONTRAM does not explictly consider progression of platoons along signalized arterials.

TABLE 3 EVALUATION OF FINAL GROUP OF MODELS

Characteristic	FREQ	CORQ (CORCON)	CONTRAM	SATURN	TRAFLO	DYNEV
Purpose	Directional freeway	Directional freeway-arterial corridor	Network of signalized and unsignalized junctions	Network of signalized and unsignalized junctions	Composite freeway-surface network	Dynamic evacuation
Source of components						
Assignment	n/a	New	New	New	TRAFFIC	TRAFFIC
Traffic flows	New	New	New	New	NETFLO I-III, FREFLO	NETFLO II
Freeway treatment						
Flow representation	Fluid	Fluid	—	—	Fluid	Flow profile
Travel time calculation	HCM (speed/volume)	Empirical step function	—	—	Conservation equations, speed-density	Speed-density
Optimization	Linear program, ramp metering	Demand responsive, ramp metering	—	—	?	—
Merging	HCM (1965)	Included	—	—	—	?
Weaving	HCR (1965)	Window provided	—	—	—	?
Traffic signal						
Evaluation	Davidson's equation	Empirical *tt*-curve	Webster's formula	Cyclic profile	Various	Flow profile
Assignment	Davidson's equation	Empirical *tt*-curve	Webster's formula	Fitted curve	BPR	BPR
Coordination	No	Empirical *tt*-curve	No	Yes	Sometimes	Yes
Self-calculation	No	No	Yes	Yes	?	Probably
Optimization	No	No	Yes (no coordination)	No	No	No
Queueing						
Spillback	Not on ramps	Yes	No	No	—	Assignment, no; simulation, yes
Holdback	Yes	Yes	Yes	Yes (queue reduction factor)	—	Assignment, no; simulation, approximation
Spillover	15 min	15 min	Continuous	15 min	—	Assignment by hand; simulation, approximation
Assignment						
Method	Diversion	Incremental/iterative	Marginal (packets)	Combination of all or nothing	TRAFFIC	Modified TRAFFIC
Freeway	—	Incremental/iterative	—	—	TRAFFIC (*34*)	TRAFFIC (*34*)
Surface	Diversion of ramp queues	Incremental/iterative	Packets	Combination of all or nothing	TRAFFIC (*34*)	TRAFFIC (*34*)

TABLE 3 *continued*

Characteristic	FREQ	CORQ (CORCON)	CONTRAM	SATURN	TRAFLO	DYNEV
Reassignment en route	Queues each time slice	Queues each time slice	Vehicle packets each packet	Queues next time slice	No	No (optional) diversion)
Reassigned O-D pattern	Next slice's destinations	Original destinations	Original destinations	Not clear	—	—
Spatial propagation	Within slice (except in queue)	Within slice (except in queue)	Traced in time	Within slice (except in queue)	Within slice	Within slice
Accessibility						
Implementation						
Mainframe	Yes	Yes	Yes	Yes	Yes	Yes
Microcomputer	No	Yes	No	No	No	No
Source code availability	Yes	Yes	—	—	No	No (proprietary)
Cost ($)						
Fixed	Negligible	0	612	525	Negligible	—
Variable	Negligible	0	2,625	4,900	Negligible	—
Support	Yes	Yes	Yes	Yes	Yes	Yes
Outputs (flows, queues, travel times)	Yes	Yes	Yes	Yes	Yes	Yes
Document						
User's manual	Yes	No	Yes	Yes	Yes	—
Theory	Yes	Yes	Yes	Yes	?	—
Modification						
Unnecessary	No	No	No	No	No	No
Feasible	Possible	Possible	Possible	Possible	Difficult	No
Comments						
Primary weaknesses	No traffic signals, no assignment	No explicit traffic signals	No freeways	No freeways	Different sub-network models, TRAFFIC (nonqueueing), 146,000 lines	Strictly NET-FLO II, TRAFFIC (nonqueueing)
Special strengths	Priority lanes, linear program optimization, priority entry	Modeling of queue spillback/ service	Sophisticated assignment technique	Coordinated signals, recalculation of signal timings	Model detail tailored	Vehicle distribution between lanes, Kalman filter capacity recalculation

NOTE: Dash indicates data unknown.

SATURN

The main strength of SATURN lies in its ability to perform assignment in a network consisting of traffic signals while giving due consideration to the specifics of the platooning structure of vehicle arrivals and the phasing of the signals. An additional feature of the model is its close linkage to a program for generating synthetic O-Ds, which is important in view of the traditional difficulties of obtaining accurate and recent O-D demands efficiently.

The main weaknesses of SATURN are its lack of a true queueing-based assignment and its lack of freeway-modeling routines. Although a number of features have been included to allow the assignment model to approximate a number of queueing impacts, it is not clear that a simple queue reduction factor accurately models all the relevant features. Specifically, in view of queueing considerations, there appear to be difficulties in terms of queue spillback, the reassignment of queues in subsequent time slices, and the use of an "equilibrium assignment technique," which employs a combination of all-or-nothing assignments. It would appear that queueing should be directly accounted for in the assignment rather than being finessed afterwards.

Although these weaknesses may not be crucial for the applications that are usually considered with SATURN, they appear to be critical in terms of the criteria specified for freeway-arterial corridors where queueing effects are a dominant factor in generating control strategies.

FINAL EVALUATION AND ELIMINATION

The final selection of models was performed by considering both the models' current capabilities and their potential to be enhanced. A detailed evaluation of the models on the short list is provided in Table 3. Although the weaknesses and deficiencies tended to drive the elimination process, any special strengths were noted for potential incorporation into the models selected for development and application.

In the final analysis, FREQ was eliminated from further consideration because it is not truly a network-based model. Although its diversion technique may be sufficient for isolated freeway corridors, when there is only one other significant parallel alternative, it is not adequate when several alternatives are possible or when the amount of diversion varies with relative flows and queues.

Because of the basic requirement that a recommended model have a queueing-based assignment technique, TRAFLO and DYNEV were eliminated. The evaluation portion of each of these models has some queueing capabilities, but the TRAFFIC assignment technique does not consider queue assignment or reassignment, and cannot easily be modified to do so.

CONTRAM appears more promising than SATURN for the type of applications that are considered in Ontario. Although neither model has provision for freeways in its current form, the flexibility of CONTRAM's assignment-evaluation-queueing technique appears to make it more suitable. In general, it appears that the model structure of SATURN may be too signal-oriented to permit the model to incorporate freeways without major fundamental changes to its assignment or queueing analysis, or both.

CONCLUSIONS

Both CONTRAM and CORQ were recommended for further development, because they appear best suited for the required modifications and have assignment techniques that can effectively deal with the dynamic growth and decay of queues in a network setting. However, modifications are required for both models, because CONTRAM, which contains a detailed treatment of traffic signals, has no freeway capabilities, whereas CORQ, which emphasizes freeways, has a weaker traffic signal base.

CONTRAM should be studied in greater detail and applied to a sample freeway-arterial corridor to determine its ability to model freeway sections and ramps. This study should identify what further enhancements the model needs and establish whether the network size constraints of CONTRAM are critical in typical freeway-arterial corridors.

Because CORQ was originally designed for freeway-arterial corridors, it automatically scored high on a number of essential requirements. However, before significant further use, the following enhancements are recommended:

- Automation of the generation of relationships of link flow versus travel time and intersection delay,
- Incorporation of a feed-forward mechanism to represent drivers' preknowledge of future network conditions,
- Improvement of link performance summaries and user documentation, and
- Consideration of the effects of spillback through signalized intersections.

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Freeway Operations and the Cusp Catastrophe: An Empirical Analysis

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Previous empirical work has shown that freeway operations sometimes result in discontinuous data and sometimes do not. Catastrophe theory has recently been proposed as one way of understanding the operations of freeways, which can account for the discontinuities in data. Data from the Queen Elizabeth Way in Ontario are used to test statistically how well the cusp catastrophe can replicate freeway operations. The results are extremely promising, with R^2-values generally above 0.7, and with results that make physical sense in terms of behavior of traffic on the roadway.

During periods of heavy traffic, freeway operations are usually characterized by sudden changes in speed as uncongested traffic encounters a bottleneck and becomes subject to stop-and-go conditions. Navin (*1*) has suggested that the cusp catastrophe from catastrophe theory (*2, 3*) can provide a way to model this sudden change. Hall (*4*) pursued this suggestion, using generalized curves based on data from the Queen Elizabeth Way (QEW) in Ontario, Canada, and found quite promising results. No attempt has yet been made, however, to fit actual traffic data to the cusp-catastrophe surface statistically. In this paper that task is accomplished: data on speeds, flows, and occupancies from four separate lane-station combinations on the QEW are used to test catastrophe theory statistically.

In the first section two elements of the background for this analysis, the empirical and the theoretical, are provided. The empirical background draws on earlier studies of the QEW data to show that both continuous and discontinuous data patterns (e.g., for flow-occupancy curves) can result from the same underlying function. In the theoretical discussion the idea behind catastrophe theory, the way the cusp catastrophe was used by both Navin and Hall, and how each of those accords with the data patterns are outlined. In the second section the transformations of the traffic operations variables necessary to have them conform with the catastrophe theory surface are dealt with. This is necessarily an extended treatment, because the range of possible transformations is large, yet only a few conform to both reality and the requirements of catastrophe theory. In the third section the results of the statistical analysis for the four lane-station combinations are presented. The final section is a discussion of some of the key points from the analysis, leading to the conclusions.

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BACKGROUND

Previous empirical work by the authors with these same data forms an important part of the background to the catastrophe theory work, not because they are the only ones to have worked on this problem (which is certainly not the case), but because they have reached different conclusions from the data than others have, and it is those conclusions that have led to the current experiment with the cusp catastrophe. The data to be used here are the same as those used in the previous analyses. Because they have been described in detail elsewhere (*5, 6*), only a brief description is needed here. The data were acquired by the Ontario Ministry of Transportation and Communications through their Freeway Traffic Management System on the QEW in Mississauga (near Toronto). The analyses have focused on 45 days of ideal conditions extracted from a record spanning 8 months. Data are available for the morning peak period for each of three lanes at each of nine stations along the roadway upstream of a major bottleneck. The four sets of data to be used in the current study are from the shoulder and median lanes at Stations 4 (4 km upstream of the bottleneck) and 7 (1.6 km upstream of the bottleneck). Figure 1 shows an example of the data from the Station 4 median lane for the three relationships of interest.

The starting point for the first analysis (*5*) was that most freeway data exhibit a gap or sparseness near capacity. The authors concluded that it is not necessary to postulate discontinuous functions to account for such a gap. Instead, gaps in the data can be accounted for by the specifics of the data collection location with respect to existing bottlenecks. Thus some data sets (collected close to the bottleneck) may show continuous curves, whereas others (considerably upstream of a bottleneck) will have large gaps. In the second analysis (*6*), these initial findings were confirmed at additional lanes and locations within the same data set. Further support was also provided for the idea that the flow-occupancy relationship is best represented by an inverted V.

The transition from uncongested to congested operations is almost always associated with a "jump," or sudden decrease in speed. There is a corresponding sudden increase in speed from the congested to the uncongested regime. These sudden changes in speed occur even though flow and occupancy exhibit a smooth and continuous change. This property—a discrete sudden change in one variable while other related variables are undergoing smooth, continuous change—is the type of behavior that catastrophe theory was developed to explain. Both Navin (*1*) and Hall (*4*) used the cusp catastrophe,

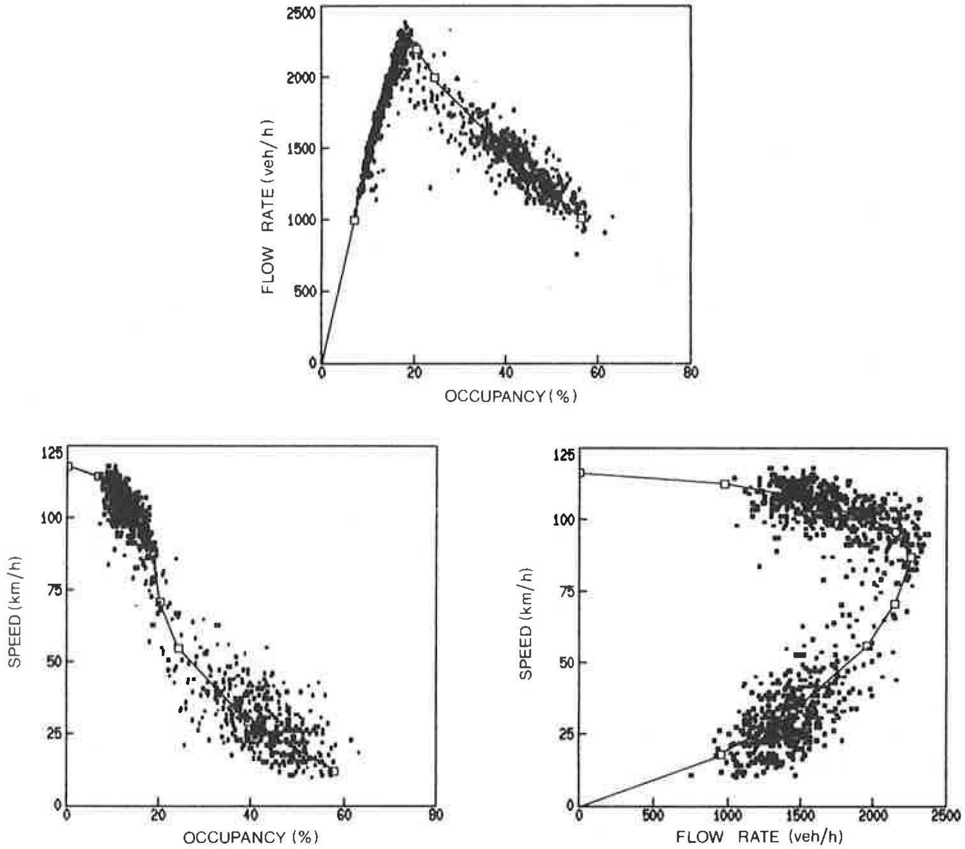

FIGURE 1 Scatter plots of 5-min averaged data from 45 days of ideal conditions, 4 km upstream of a bottleneck (lines represent approximate shapes of curves).

which is one of the seven fundamental catastrophes, so it is the only one to be discussed here.

The cusp catastrophe is described as one that minimizes a potential function,

$$V(x) = x^4 + ux^2 + vx \tag{1}$$

The critical points of the cusp catastrophe are defined by the surface:

$$4x^3 + 2ux + v = 0 \tag{2}$$

where x is referred to as the state variable—the one that sometimes exhibits the discontinuous behavior—and u and v are referred to as the control variables. A plot of the resulting partly folded surface is shown in the central part of Figure 2. The projection of the edges of the fold onto the u-v-plane (immediately below the folded surface in Figure 2) forms a cusp, which is the source of the name for this catastrophe. This cusp can be obtained by setting the discriminant of Equation 2 to zero:

$$8u^3 + 27v^2 = 0 \tag{3}$$

If this discriminant is less than zero (a point "inside" the cusp), there are three real roots, or three possible values of x. If it is greater than zero ("outside" the cusp), there is only one real root and therefore only one possible value for x. The similarity between this cusp-shaped projection and the flow-occupancy curves derived in earlier work (*5, 6*) provided the initial rationale for considering the cusp catastrophe after it had been drawn to the authors' attention by discussions with Navin.

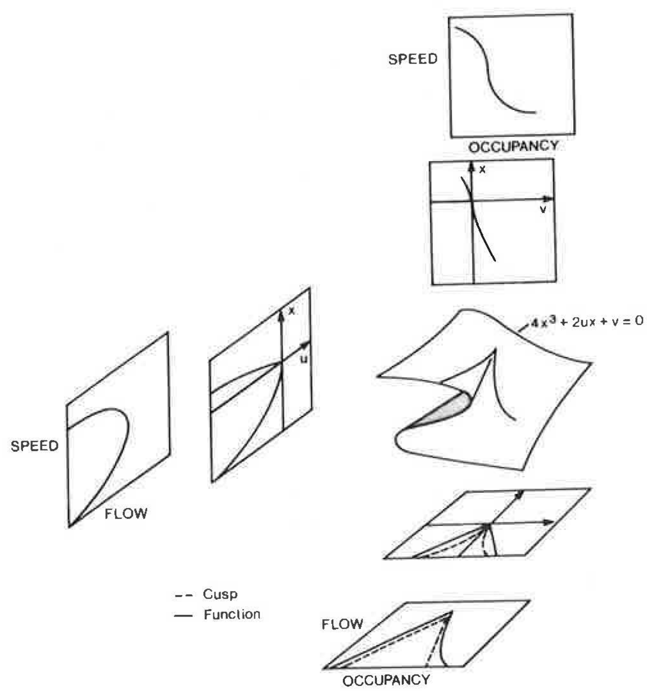

FIGURE 2 Catastrophe theory surface along with u-v, x-v, and x-u projections and corresponding traffic plots.

Surprisingly, this use of the cusp catastrophe is at variance with Navin's by 180 degrees. Because he emphasizes the discontinuity in the data, he draws the flow-concentration curve (for example) so that its high-flow end crosses the open portion of the fold, thus ensuring a discontinuity in all data sets. The use of catastrophe theory here was intended to provide a situation in which there would sometimes be continuous data and other times a gap in the data. Consequently, the authors drew the flow-concentration curve so that the tip of it crosses just above the point on the surface where the fold disappears. For the moment, both versions are merely conjectures, intended to match certain aspects of observed freeway behavior but not yet tested rigorously. One aim of this paper is to provide a more rigorous test of one version of the application of the cusp catastrophe by statistically fitting a curve to the data.

The catastrophe theory variables u, v, and x are not directly equivalent to their corresponding traffic variables but must be first subjected to some transformations. Nor is it self-evident which traffic variable corresponds with which of the three variables describing the catastrophe theory surface. Because of the discontinuities in speed and the similarity between the flow-occupancy plot and the cusp, previous efforts by Hall (*4*) chose to have x correspond with speed, u with flow, and v with occupancy. (Quite independently, Navin had made the same decisions.) It was also decided that the point where the fold disappears [i.e., the origin of the axes, point (0,0,0)] should correspond to operation at capacity. Figure 2 shows the folded surface along with the projection of a possible function onto the three planes and the transformations from these planes into the respective traffic operations plots. The anticipated location of the cusp relative to the function is also shown.

Numerous sets of transformations are possible; two were discussed in the previous paper (*4*). The first set, which serves as the starting point for this paper, is based on a simple linear transformation of speed to x and flow to u. The transformation between v and occupancy was obtained in an ad hoc fashion, using the averaged values of occupancy and v calculated with Equation 2. However, if the data points themselves are used instead, the resulting v-occupancy graph becomes a scatter plot of points that can be fitted to a third transformation. How well this third transformation fits the scatter plot then provides an indication of how well catastrophe theory models freeway operation. That, in essence, is the approach taken in this paper.

TRANSFORMATIONS

In order to determine statistically how well the actual traffic data fit the catastrophe theory surface, it is first necessary to identify the "best" set of transformations between traffic and catastrophe theory variables. The criteria for identifying a best set were established after considerable trial and error in working with possible transformations. However, for clarity the criteria are presented first and then results of the various transformations are discussed.

There are two main criteria for an appropriate set of transformations between traffic variables and the catastrophe theory surface. The first is that there be a discontinuity in speed, because all four data sets to be investigated show one. This requires that the resulting function in x-u-v space occur within that portion of the space where $u < 0$, that is, where the fold occurs.

The second criterion is that at the discontinuity, the physical behavior of the traffic operations be consistent with the mathematical behavior of the cusp catastrophe. This is perhaps best explained by diagrams. Figure 3*a* shows the full surface for the cusp catastrophe, as given by Equation 2. In Figure 3*b* the center fold has been removed, because it in fact corresponds to maxima of the function. Only the upper and lower sheets of the folded area are minima. One possible form of transition from upper to lower surface is sketched on the surfaces in Figure 3*b*: the operations remain on one sheet until it disappears. This is referred to as the perfect delay convention. The most plausible alternative is the Maxwell convention, in which the function takes on its global minimum at all times, resulting in the surface of possible values shown in Figure 3*c*. With this convention, all transitions, up or down, occur along the same plane, that where $v = 0$.

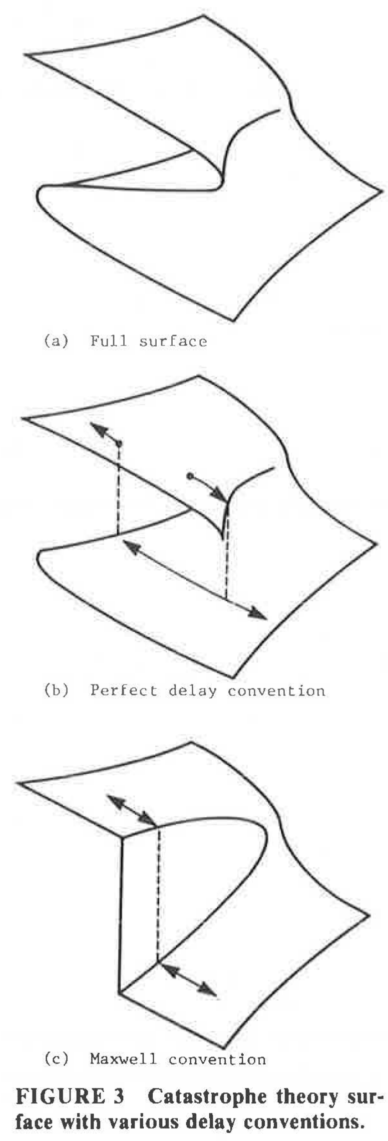

FIGURE 3 Catastrophe theory surface with various delay conventions.

Matching the physical behavior of the traffic operations against these two delay conventions produces two acceptable patterns of data once they have been transformed to the catastrophe theory variables. The perfect delay convention requires that all the uncongested data [data whose speed was greater than the speed at capacity (*SPCAP*)] remain on the top surface, to the left of the right-hand cusp on the u-v-plane. The congested data (speed less than *SPCAP*) must be on the lower surface, to the right of the left-hand cusp. Thus the second criterion translates into a specification of the location of the data on the surface. For ease of plotting and comprehension, the projection of the data onto the u-v-plane will be used to identify data locations for the different sets of transformations.

Three transformations are needed for each set: speed to x, flow to u, and occupancy to v. Given Equation 2 and the actual traffic data, any two determine the third one. The choice of the x-speed and u-flow transformations as the initial ones was arbitrary.

The transformations found to give reasonable results in the previous paper were

$$x = \text{speed} - SPCAP \tag{4}$$
$$u = (\text{flow} - \text{capacity})/1{,}000 \tag{5}$$

One way in which those results did not correspond with expectations, however, was that the projection of the cusp onto the u-v-plane was not distinguishable from the negative portion of the u-axis. Closer inspection of those results as part of this work showed that the reason is one of scale. Using the transformations given in Equations 4 and 5, typical values for x ranged from 25 to -70, whereas the corresponding values for u were in the range of 0 to -1.5. To calculate v (Equation 2), the value of x is cubed, whereas u enters only to the first power, so that v is essentially equal to $-4x^3$ and therefore goes as high as 2×10^6. However, the values of v for the cusp depend only on the value of u (Equation 3), and were therefore no greater than 2.

In order to makes the values of v for the data more comparable with those for the cusp, the effect of u on v must be increased or the effect of x must be decreased, or both. Reducing x will greatly reduce the value of v for the data points (because x is cubed in the calculation of v); however, it will not change the cusp. Increasing u will increase the size of the cusp (because, from Equation 3, u is cubed in the calculation of the cusp) but will not have much of an effect on the data points because that calculation is still dominated by x (assuming that x is still given as in Equation 4).

There are essentially two extreme cases between which the proper transformations should lie. In the first case, flows are divided by a very large number, so that u has been eliminated (i.e., $u = 0$): the data would fall on the line $v = -4x^3$. In the second case, the effect of x has been eliminated (by dividing the speed transformation by a very large number) and the data would fall on the negative u-axis. (If $x = 0$, then from Equation 2, v must equal zero as well.) The location of the data on the catastrophe theory surface for these two extreme cases is shown in Figure 4.

The first extreme, setting u equal to zero, is not desirable because it effectively prevents x (and therefore the speed) from exhibiting the discontinuous behavior that was the initial attraction of catastrophe theory. The second extreme, setting x equal to zero, is also unacceptable because that would require the data to lie on the middle surface, which Figure 3 shows is not possible. The original transformations are quite close to the first extreme: to see whether the relationship between the transformed data and the cusp can be made more obvious, other transformations are attempted. In doing so, however, it appears that the data location will migrate across the surface and around the fold, and finally both portions of the data will converge on the line $v = x = 0$. Any of the transformations that place data on the middle fold are inconsistent with the mathematics of catastrophe theory, and therefore will be rejected.

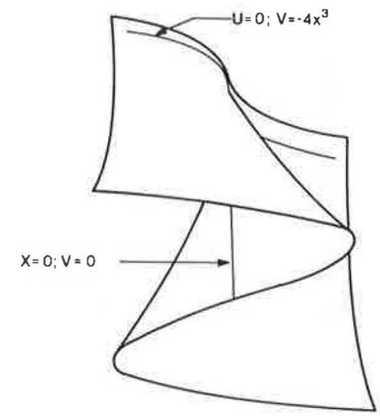

FIGURE 4 Location of traffic data on catastrophe theory surface for extreme transformations.

This migration of data can be seen in the range of u-v-plots shown in Figures 5-13, which contain the results of the first-pass search for better transformations of speed and flow. [In Figures 5-13, circles represent the uncongested data (speed less than *SPCAP*); triangles represent congested data (speed greater than *SPCAP*).] The analysis for this section is based only on the data from the median lane at Station 4 (MED4), because it would be too confusing to attempt all transformations on all four data sets. The basic transformations in Figures 5-13 were assumed to be of the following form:

$$u_1 = \text{flow} - \text{capacity} \tag{6}$$
$$x_1 = \text{speed} - SPCAP \tag{7}$$

From these, nine sets of transformations were obtained by setting $x = x_1/i$ and $u = u_1/j$, where i equals 1, 5, or 10 and j equals 1, 10, or 1,000. Selection of the best of these nine sets of transformations as the area for more detailed search was based on the following reasoning.

Figure 5 corresponds to the transformations given in Equations 4 and 5. As noted earlier, the cusp is indistinguishable from the negative u-axis. The uncongested data appear to be concentrated slightly to the left of the negative u-axis, and the congested data are widely scattered to the right of the axis. Thus this pair of transformations meets the criteria set out earlier, but is perhaps too close to a line for which $u = 0$. The situation is not improved much by dividing x_1 by larger

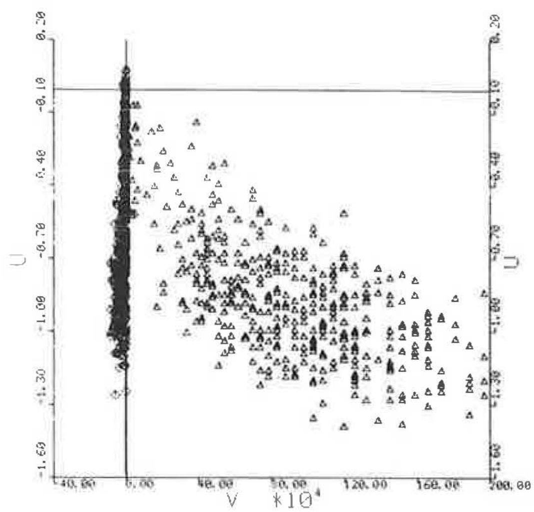

FIGURE 5 Plot of u versus v and cusp projection, MED4: $u = u_1/1{,}000$; $x = x_1/1$.

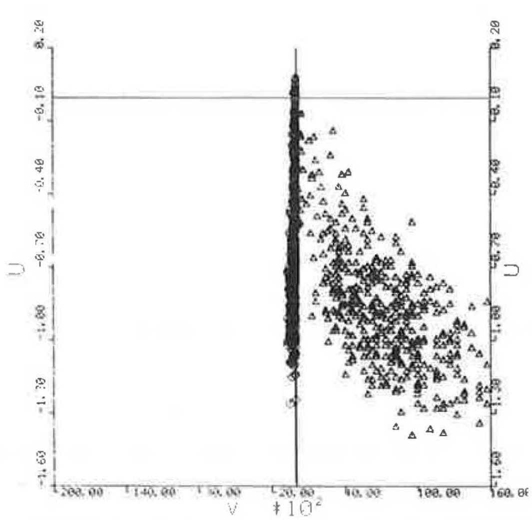

FIGURE 6 Plot of u versus v and cusp projection, MED4: $u = u_1/1{,}000$; $x = x_1/5$.

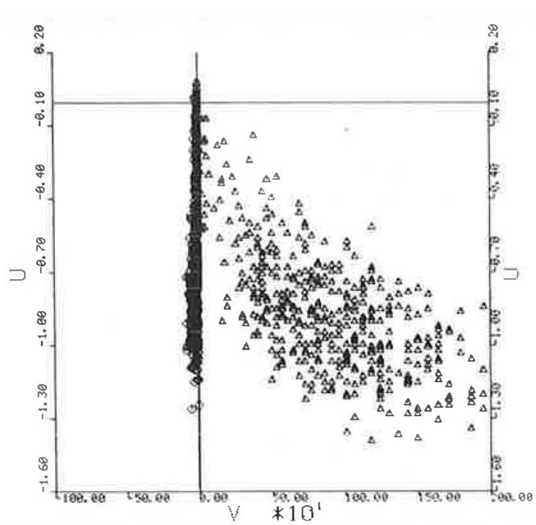

FIGURE 7 Plot of u versus v and cusp projection, MED4: $u = u_1/1{,}000$; $x = x_1/10$.

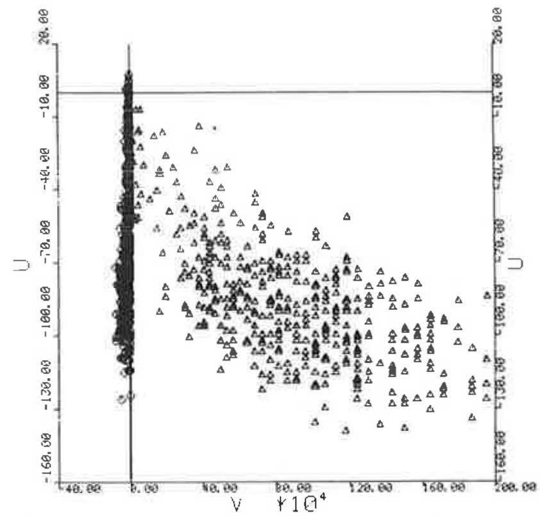

FIGURE 8 Plot of u versus v and cusp projection, MED4: $u = u_1/10$; $x = x_1/1$.

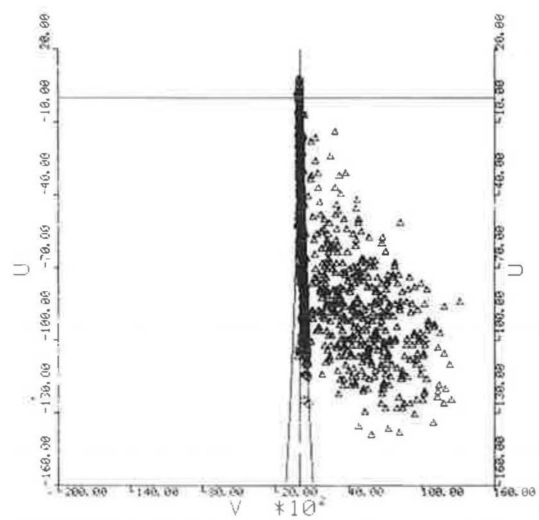

FIGURE 9 Plot of u versus v and cusp projection, MED4: $u = u_1/10$; $x = x_1/5$.

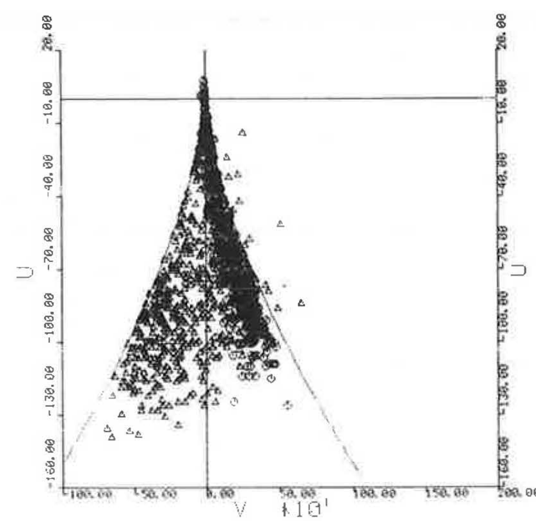

FIGURE 10 Plot of u versus v and cusp projection, MED4: $u = u_1/10$; $x = x_1/10$.

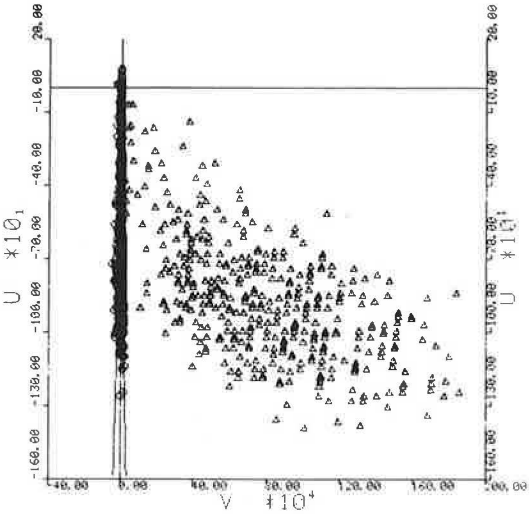

FIGURE 11 Plot of u versus v and cusp projection, MED4: $u = u_1/1$; $x = x_1/1$.

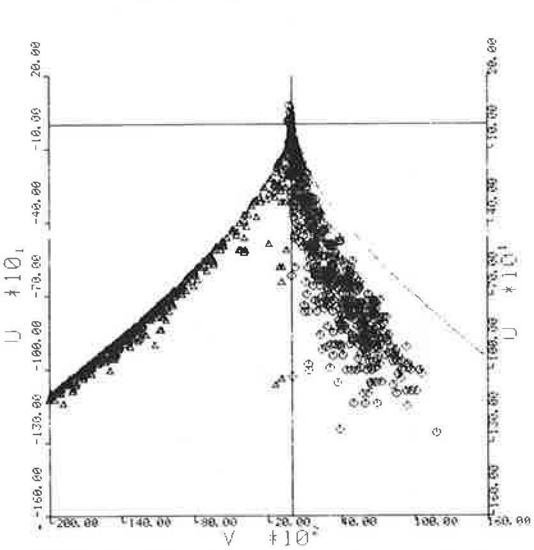

FIGURE 12 Plot of u versus v and cusp projection, MED4: $u = u_1/1$; $x = x_1/5$.

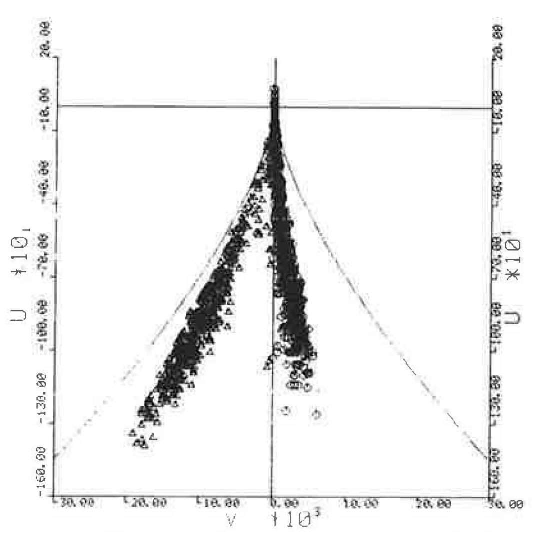

FIGURE 13 Plot of u versus v and cusp projection, MED4: $u = u_1/1$; $x = x_1/10$.

numbers, thereby decreasing the importance of x (Figures 6 and 7).

As u_1 is divided by smaller numbers, such that the effect of u is increased (Figures 8 and 11), there appears to be little effect on the relative locations of the congested and uncongested data and only a minimal effect on the size of the cusp. However, closer examination of the uncongested data (Figures 14-16) shows that increasing the effect of u while keeping x constant causes the uncongested data to migrate, first into the cusp and then across the negative u-axis, and finally to cluster along the right-hand cusp boundary.

When the effects of both x and u are varied simultaneously by dividing x_1 by larger numbers and u_1 by smaller ones, relative to Equations 4 and 5, the migration of the congested data becomes apparent (Figures 8-10). These same figures show migration of the uncongested data onto the center surface. In Figure 9, the concentration of data along the right-hand cusp corresponds to uncongested data that have migrated as far to the right as possible. In Figure 10, the uncongested data have moved away from the cusp again.

Data inside the cusp could fall on any one of the three surfaces, and it is not clear from the u-v-plots alone which one of the surfaces they are on. In order to clarify where the data appear on the surface, a short BASIC program was written that plotted the folded catastrophe theory surface and allowed the user to move a cursor along the surface. The cursor's coordinates in the x, u, v-space were displayed so that the user could position in cursor at appropriate data point locations. Using this interactive plotting routine, it was determined that the uncongested data in Figure 10 fall on the middle surface.

This same procedure was used to confirm that the migration of the uncongested data continues still further toward the $v = 0$ line, on the middle surface, in Figures 12 and 13. The congested data have also continued their migration in these two figures, first appearing to be trapped against the left-hand boundary of the cusp, then moving away from it again,

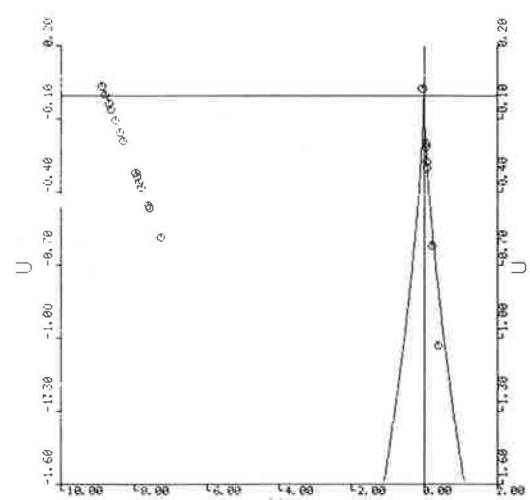

FIGURE 14 Enlargement of u-v-plot (uncongested data only) and cusp projection, MED4: $u = u_1/1{,}000$; $x = x_1/1$.

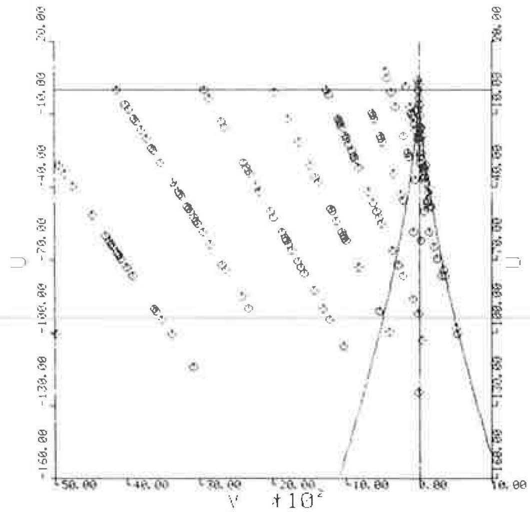

FIGURE 15 Enlargement of *u*-*v*-plot (uncongested data only) and cusp projection, MED4: $u = u_1/10$; $x = x_1/1$.

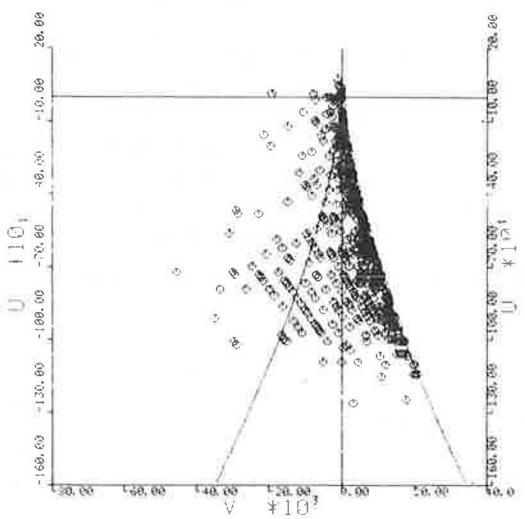

FIGURE 16 Enlargement of *u*-*v*-plot (uncongested data only) and cusp projection, MED4: $u = u_1/1$; $x = x_1/1$.

but on the middle surface. There are also some data on the middle surface in Figures 9 and 11.

The conclusion from this first series of transformations, then, is that only Figure 8 contains reasonable results, and further inspection of transformations between it and Figure 5 is warranted. From this further inspection, the following transformations were determined to be the best (given the time available):

$$u = (\text{flow - capacity})/100 \qquad (8)$$

$$x = \text{speed} - SPCAP \qquad (9)$$

The overall data pattern for the best transformation found is shown in Figure 17. When the cusp and the uncongested data in its vicinity are inspected at a much larger scale (Figure 18), it is found that only 11 data points (out of 478 for uncongested operations) occur across the plane that would define the Maxwell convention. (By definition, none can be out of place for the perfect delay convention.) This was deemed to be acceptable for empirical research, so these are the transformations that are used to estimate the statistical fit.

STATISTICAL ANALYSIS

The purpose of this section is to determine statistically how well the transformations in Equations 8 and 9 fit the data to the catastrophe theory surface. For each data point, the value of v, obtained by using Equations 8, 9, and 2, was plotted against the corresponding occupancy (*OCC*); thus the v-*OCC* plot was obtained for each of the four lane-station combinations (Figures 19-24). In Figures 19-24 and Table 1, the median and shoulder lanes of Stations 4 and 7 are referred to as MED4, SHL4, MED7, and SHL7, respectively. Both median lanes combined are referred to as MED and both shoulder lanes combined as SHL.

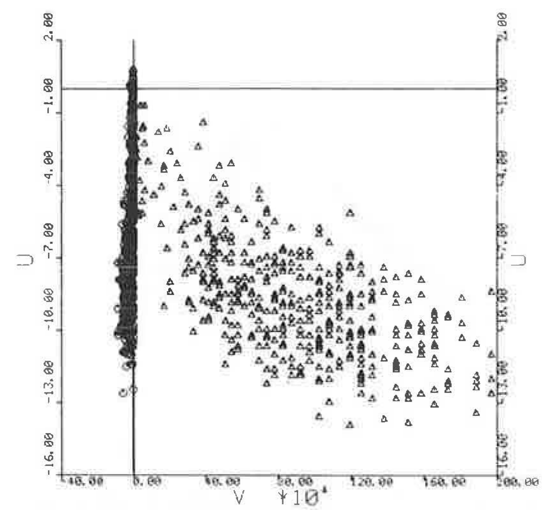

FIGURE 17 Plot of u versus v and cusp projection, MED4: $u = u_1/100$; $x = x_1/1$.

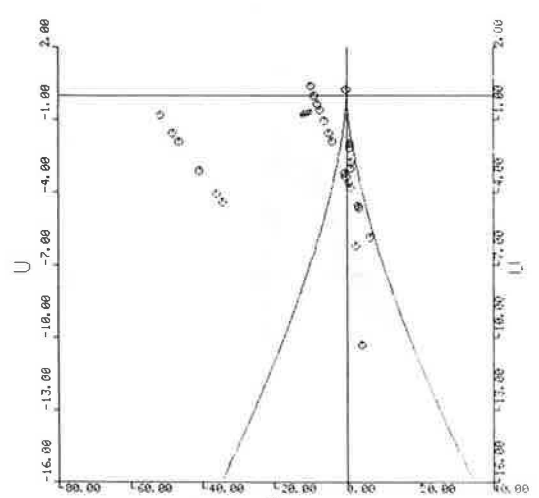

FIGURE 18 Plot of u versus v and cusp projection, MED4: $u = u_1/100$; $x = x_1/1$ (uncongested data only).

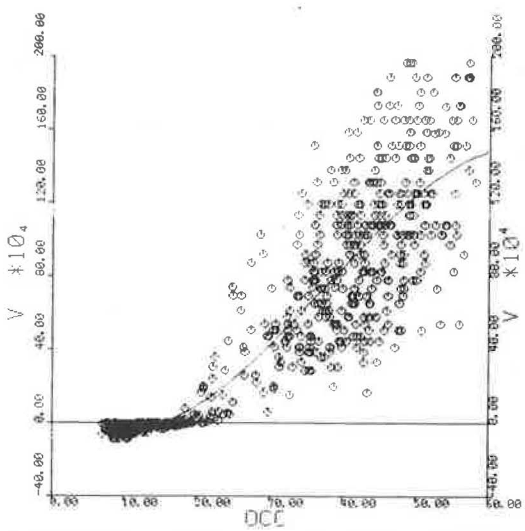

FIGURE 19 *v-OCC* **plot and best-fit line: MED4.**

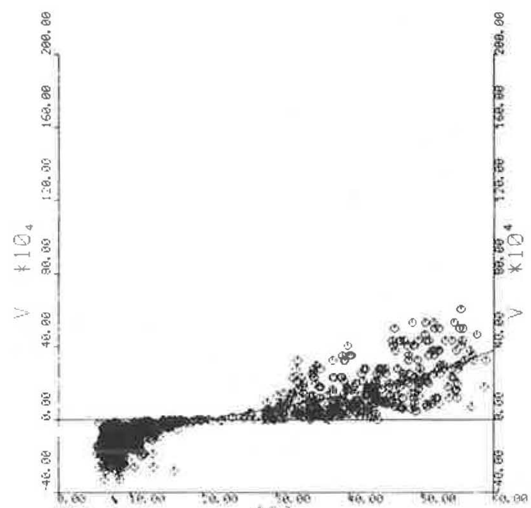

FIGURE 20 *v-OCC* **plot and best-fit line: SHL4.**

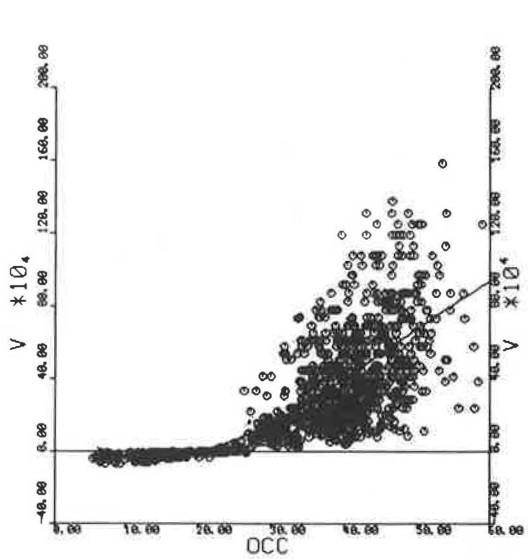

FIGURE 21 *v-OCC* **plot and best-fit line: MED7.**

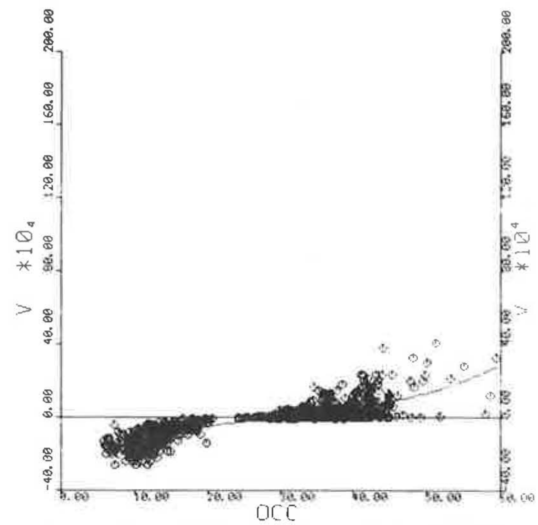

FIGURE 22 *v-OCC* **plot and best-fit line: SHL7.**

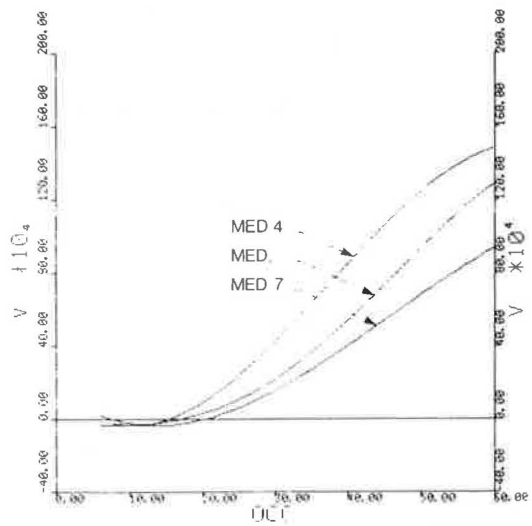

FIGURE 23 *v-OCC* **plot and best-fit line: MED4, MED7, and MED.**

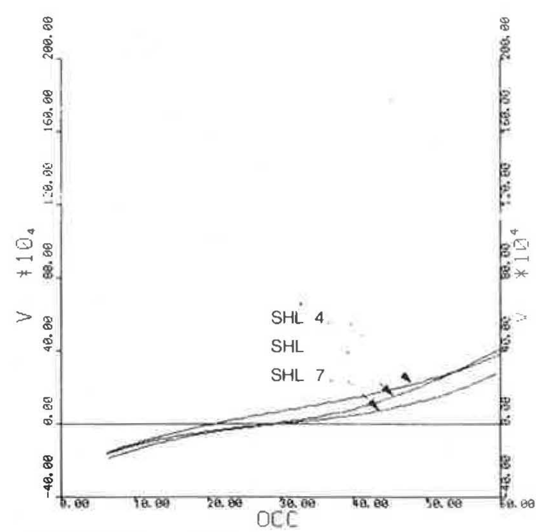

FIGURE 24 *v-OCC* **plot and best-fit line: SHL4, SHL7, and SHL.**

TABLE 1 RESULTS OF STATISTICAL ANALYSIS

Lane-Station Combination	No. of Data Points	R^{2a}
MED4	950	0.839
MED7	1,096	0.586
MED	2,046	0.634
SHL4	882	0.785
SHL4[b]	882	0.786
SHL7	1,117	0.730
SHL	1,999	0.723
SHL[b]	1,999	0.728

[a]For the cubic equation:
$v = a_0 + a_1(OCC)^1 + a_2(OCC)^2 + a_3(OCC)^3$
[b]Results for the quatric equation:
$v = a_0 + a_1(OCC)^1 + a_2(OCC)^2 + a_3(OCC)^3 + a_4(OCC)^4$

The main task was to determine what function, if any, could be used to describe the relationship between v and OCC and how well this function fit the data statistically. Numerous different types of functions were contemplated, but it was finally decided that a simple polynomial in OCC can best describe v. In other words, the general form of the relationship would be

$$v = a_0 + a_1(OCC)^1 + a_2(OCC)^2 + \ldots + a_n(OCC)^n \quad (10)$$

A polynomial regression routine (routine RLFOR of the International Mathematical and Statistical Libraries, Inc.) was utilized to determine the best polynomial equation to fit the data for each lane-station combination. The routine was also used to determine the maximum degree of the polynomial, that is, what the value of n is in Equation 10. A sequential, or stepwise, procedure utilizing partial F-values was used to determine how many terms the polynomial should have. It was found that a cubic ($n = 3$) was sufficient in explaining the v-OCC relationship for nearly all of the lane-station combinations. (The shoulder lane of Station 4 and both shoulder lanes combined were explained better by a quatric ($n = 4$) in OCC; however, for consistency, the cubic was used throughout.) A summary of the results for the four separate lane-station combinations as well as both median lanes and both shoulder lanes can be found in Table 1. The best-fit lines along with the v-OCC scatter plots for the four lane-station combinations are shown in Figures 19-24.

The results are promising. Most of the lane-station combinations had R^2-values of more than 0.7. Also, from the plots of the best-fit lines, it appears that the lines cross the $v = 0$ axis near the point of critical occupancy.

However, these results need to be interpreted with considerable caution. The v-OCC plots show that the data are heteroscedastic (especially in the median lanes): there is a larger variance for the higher occupancies (congested regime) than for the lower ones (uncongested regime). The presence of heteroscedasticity means that the regression coefficients may be biased estimates. There are two potential explanations for the occurrence of this problem. The first rests on the lack of precision of the measurements underlying v, which is highly dependent on x, which in turn is calculated solely from speed. The measurement of speed is imprecise in the congested regime because of the stop-and-go nature of the traffic and the 5-min time intervals used for the data acquisition.

The second explanation follows from the observation that the heteroscedasticity is not as apparent in the shoulder lanes as it is in the median lanes. For the median lanes, $SPCAP$ was estimated to be approximately 90 km/hr, which is clearly closer to speed during uncongested operation (typically 100 to 115 km/hr) than that during congested operation (20 to 40 km/hr). This difference between observed speed and $SPCAP$ is cubed in the calculation of v, thus leading to larger variances for larger values of x and v. For the shoulder lanes, $SPCAP$ was estimated to be about 60 km/hr, which is more centrally located between speeds in uncongested operation (typically 70 to 90 km/hr) and those in the congested regime (30 to 50 km/hr). Therefore, the variance in the congested and uncongested regimes would be more nearly equal, as Figures 19-24 confirm.

Because $SPCAP$ could not be identified with any certainty (it occurs where the gap in the data does), it seemed appropriate to investigate briefly the consequences of using other values for this key variable. Again, the median lane at Station 4 was used, in part because it shows the most scatter in Figures 19-24, in part because it was the one investigated in detail to develop the initial transformations. Critical speeds of 75, 80, and 85 km/hr were used, and plots of the u-v and v-OCC diagrams were created (Figures 25-30). These show that the use of a lower value for $SPCAP$ causes the uncongested data to move farther to the left of the cusp (Figures 25, 27, and 29). As is apparent from the v-OCC plots in Figures 26, 28, and 30, the disparity in variance between congested (negative v) and uncongested regimes has improved slightly by decreasing $SPCAP$. However, a lower R^2 was obtained for the lower critical speeds. This is most likely caused by the fact that lowering $SPCAP$ while lowering the variance in the congested data also increases the variance for uncongested operations. Because there are more uncongested than congested data from this lane-station combination and because the uncongested data appear to be more clustered (in terms of occupancy), any increase in the variance of the uncongested data will have an adverse effect on the fit. Consequently, the original estimates of $SPCAP$ have been retained.

DISCUSSION AND CONCLUSIONS

The overall results of this effort to apply catastrophe theory to traffic operations data appear to be very promising. The data can be transformed in such a manner that they conform to both catastrophe theory and the physical reality of traffic operations. The statistical analysis appears to be promising in that a generally good fit was obtained in regressing the v-OCC data to a polynomial in OCC for the four lane-station combinations studied. There are, however, problems that have arisen in this first effort to fit data to theory. Four main ones are worthy of note. Two have been raised already, but not resolved: the heteroscedasticity of the data for the v-OCC plot and the identification of $SPCAP$. The other two are more basic, and have not yet been raised. One relates to the generality of results with respect to different lanes and

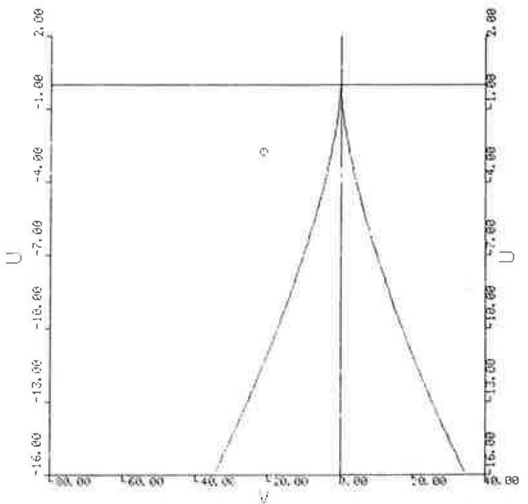

FIGURE 25 Plot of *u* versus *v* (uncongested data only) with cusp projection for best MED4 transformation: *SPCAP* = 75 km/hr.

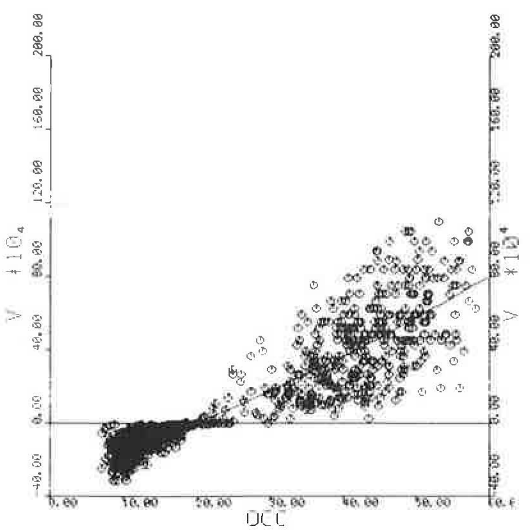

FIGURE 26 *v-OCC* plot (all data) for best MED4 transformation: *SPCAP* = 75 km/hr.

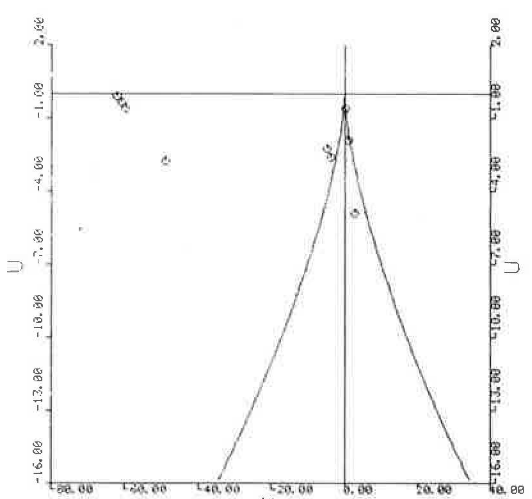

FIGURE 27 Plot of *u* versus *v* (uncongested data only) with cusp projection for best MED4 transformation: *SPCAP* = 80 km/hr.

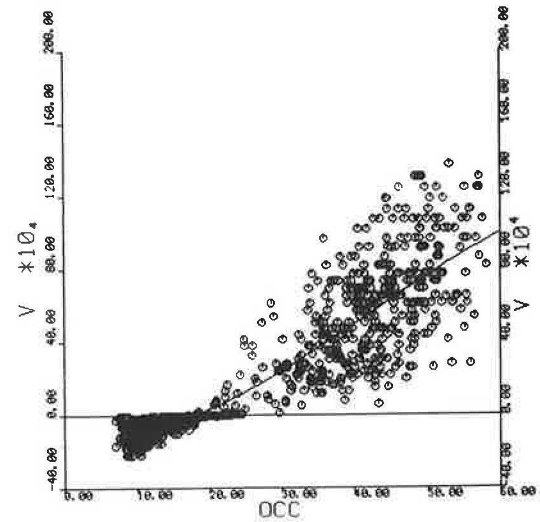

FIGURE 28 *v-OCC* plot (all data) for best MED4 transformation: *SPCAP* = 80 km/hr.

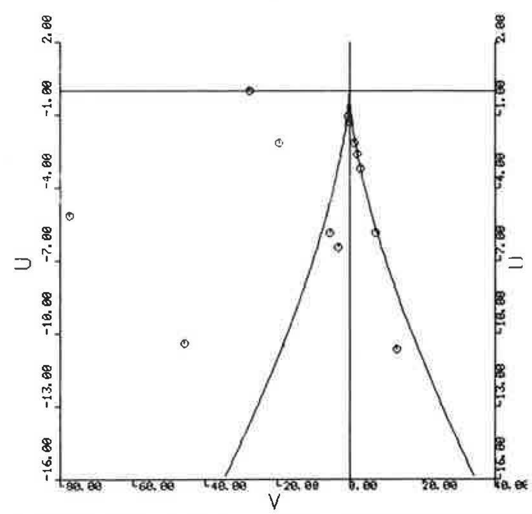

FIGURE 29 Plot of *u* versus *v* (uncongested data only) with cusp projection for best MED4 transformation: *SPCAP* = 85 km/hr.

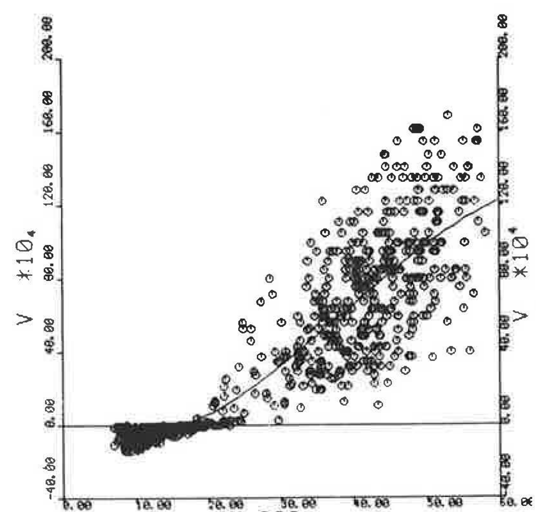

FIGURE 30 *v-OCC* plot (all data) for best MED4 transformation: *SPCAP* = 85 km/hr.

stations—should one be trying to fit each separately, or expect the same parameters to apply to many places? The second addresses the mechanics of conducting the transformations—what might happen, for example, if v were calculated first, rather than x?

There was clearly some heteroscedasticity in the data, especially for the median lanes. There are several standard ways to overcome this, which time did not permit to be included in this paper. Perhaps the polynomial should not be regressed in terms of OCC, but rather in terms of $1/OCC$ or $\exp(OCC)$ (*7*). Alternatively, as mentioned in the previous section, the selection of *SPCAP* may affect the variance of the error, although the results (Figures 25-30) did not provide very strong support for that approach.

Identification of *SPCAP* for this paper was essentially estimated from the speed-flow plots. Clearly this identification affects the goodness of the fit, and therefore should be included in the optimizing procedures.

Yet before that is done, a decision should be made regarding the desired or expected generality of the v-OCC relationship, or indeed of all of the transformations. For these analyses, the same general transformation was used for all lanes and locations, but different values were allowed in each case for flow and *SPCAP*. Is that reasonable, or would it be better to select a single value that can serve for all locations? Likewise, should the third transformation determined statistically be the same for all lane-station combinations or different in each case? Alternatively, there could be one set of transformations for median lanes and a different set for shoulder lanes regardless of location.

The solution of this issue is not entirely a matter for curvefitting and statistical analysis, although they can help to resolve it. Earlier work (*6*) demonstrated that there are distinct differences in the operational characteristics of the shoulder and median lanes, particularly in the values of the flows and *SPCAP* but possibly also in the general shape of the curves. This finding leads one to expect that a common transformation would not be desirable, but that there should be separate transformations for the shoulder and the median lanes. However, the two median-lane results shown here in Table 1 and Figure 8 are not particularly similar. Whether they would be improved with some other value of *SPCAP* appears unlikely.

The final issue to be raised is the mechanics of conducting the transformations. It is possible to begin with transformations of flow to u and occupancy to v, and from them to calculate x. Then the final step would be to fit a transformation between x and speed. A few preliminary efforts in that direction suggest that the behavior of the data with respect to the cusp can be much better controlled, but that calculation of x (as a cube root) is more difficult to control. This approach may or may not help the heteroscedasticity problem as well. Time and space limitations precluded the extension of the work at this time, but it appears potentially both interesting and valuable. Among other things, it would permit the prediction of speeds on the basis of flow and occupancy, which would be of considerable practical importance to those freeway systems with single rather than paired detectors at each station.

In this paper it has been demonstrated that catastrophe theory can describe freeway operations with a reasonable amount of statistical precision. However, it has also been shown that there are numerous areas where future work could be carried out. The most important point is that the cusp catastrophe provides a valuable new way to understand freeway operations as they move in and out of congested conditions, and therefore may provide a better tool for management of freeway systems.

ACKNOWLEDGMENTS

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Delay Analysis for Freeway Corridor Surveillance, Communication, and Control Systems

B. RAY DERR

A method of estimating the delay savings due to installation of a freeway corridor surveillance, communication, and control (SC&C) system is discussed. Using reasonable assumptions, the model estimates recurrent delay by speed-flow relationships and includes the effects of diversion to the frontage road. The nonrecurrent delay savings is found by using a graphical technique on a plot of time versus cumulative vehicles. The model parameters are easily adjusted for local conditions. The model provides a valuable tool for ranking SC&C projects and obtaining an estimate of their benefits.

In a research project conducted by the Texas Transportation Institute for the Texas Department of Highways and Public Transportation and completed in 1983 a methodology was developed for estimating the delay savings due to geometric improvements to an existing highway (*1*). Using a 20-year life, this model provides a cost-benefit ratio for a highway project and is currently being used in Texas to rank construction projects by priority.

A separate model developed to analyze freeway corridor surveillance, communication, and control (SC&C) systems is discussed here. Like the geometric model, the primary intent of this model is to provide an objective method of ranking construction projects and it contains many of the basic assumptions of the geometric model to ensure compatible results. The model requires only data that are commonly available in the planning stage of a project.

An SC&C system helps to provide safer and more efficient traffic operation by monitoring the traffic flow and, in case of congestion, controlling the traffic and helping clear the congestion. The principal elements of an SC&C system are surveillance loops and cameras, ramp metering, and a responsive signal system. The surveillance loops and cameras serve to alert an operator to a congested condition on the freeway. The operator can then use the ramp metering to divert traffic off the freeway onto the frontage road (or parallel street). In order to handle this increased traffic, the signal timings along the frontage road are adjusted to increase capacity and minimize delay.

The model calculates both the recurrent and the nonrecurrent delay saved by an SC&C system. Recurrent delay occurs every day at the same time of day and for the same length of time, barring normal statistical differences from day to day. The chief examples of recurrent delay are the morning and evening peak hours. Nonrecurrent delay is caused by an incident that is not expected, generally an accident or a stalled vehicle. Studies in California indicate that nonrecurrent delay can exceed the recurrent delay on a typical freeway (*2*).

Inputs to the model include the present and 20th-year average daily traffic (ADT), the number of lanes on the freeway and frontage road, the length of the project, and the cost of the SC&C system. The ADT is taken at a typical spot in the freeway section and ramp locations, and volumes are not handled explicitly. The model uses these very gross inputs to estimate the delay savings due to an SC&C system.

GENERAL OPERATION

Using the present and 20th-year ADT, an ADT for each of the 20 years of the project's life is found by using a logarithmic-type growth curve (*1*).

$$ADT_t = ADT_c(t + 1)^e$$

where

ADT_t = estimated ADT for year t;
ADT_c = current ADT, year 1;
e = $[\ln(ADT_p) - \ln(ADT_c)] / \ln(T)$;
ADT_p = projected ADT for year T; and
T = year at end of planning horizon = 20.

In finding the recurrent delay, typical K-factors for urban Texas freeways are used to determine the hourly volumes from the ADT (*1*). The directional distribution is assumed to be 50 percent. The hourly traffic is then distributed between the frontage road and the freeway, and average speeds are found for each by using some speed-flow relationships. If a queue exists at the end of the hour, it is carried over into the next hour. The traffic volume is multiplied by the length of the freeway section and divided by the average speed to obtain the total travel time. This is done for both the uncontrolled and the controlled freeway and the difference represents the recurrent delay savings for the day. To find the

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nonrecurrent delay, an analytical version of the graphical method shown in Figure 1 is used as described later (3). Once again, the delay is found for both the uncontrolled and the controlled freeways, and the nonrecurrent delay savings is the difference.

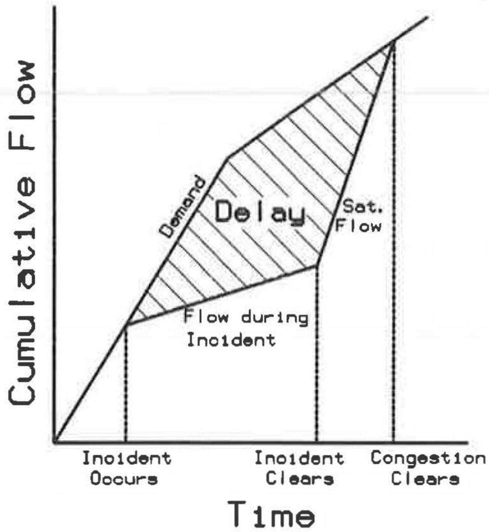

FIGURE 1 Nonrecurrent delay.

The total daily delay savings for each of the 20 years is then factored up to an annual savings. These 20 annual delay savings are then converted to monetary values and discounted to obtain the present value of the delay savings. Dividing by the cost of the system produces a benefit-cost ratio. The following sections give a further description of how the model works.

DISTRIBUTION OF TRAFFIC

The following parameters and variables are used:

q_T	=	total hourly flow [vehicles per hour (vph)],
q_M	=	flow + average queue on main lanes (vph),
q_F	=	flow on frontage road (vph),
s_0	=	queue at beginning of the hour (vehicles),
s_1	=	queue at end of the hour (vehicles),
N_M	=	number of main lanes,
N_F	=	number of frontage-road lanes,
Q_M	=	main-lane capacity [vph per lane (vphpl)],
Q_B	=	main-lane capacity after breakdown (vphpl),
Q_F	=	frontage-road capacity (vphpl),
Q_{FB}	=	frontage-road volume during main-lane breakdown (vphpl),
δQ_M	=	main-lane volume at which diversion to frontage road begins (vphpl),
γQ_B	=	used after main-lane breakdown; main-lane volume + average queue above which frontage road is fully loaded (vphpl), and
θQ_B	=	used after main-lane breakdown; main-lane volume + average queue at which there would be no vehicles on frontage road (vphpl).

Given the hourly traffic determined by the ADT and the assumed K-factors, the first step to finding the recurrent delay is to distribute the traffic between the freeway and the frontage road. Typical diversion curves used are shown in Figure 2. Basically, the freeway will carry all the traffic until the hourly volume reaches a certain point ($N_M \delta Q_M$). Between this point and until the main lanes reach capacity ($N_M Q_M$), the frontage road will carry an increasing portion of the total traffic. When the main lanes reach capacity, the main-lane breakdown and a new set of curves are used.

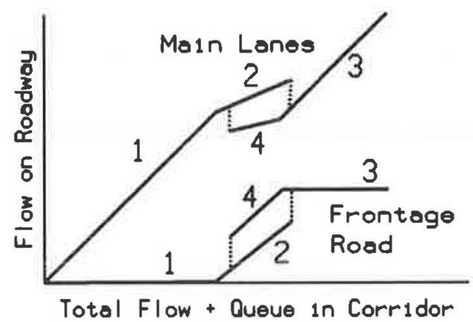

FIGURE 2 Typical diversion curves.

Rather than the hourly flow, the flow plus average queue is used after breakdown to account for the effect of a standing queue (1). As long as the flow plus average queue stays above $N_M \gamma Q_B + N_F Q_F$, both the main lanes and the frontage road will be operating at capacity. If the main-lane flow falls below $N_M Q_B$, the main lanes will recover and the other set of diversion curves will be used. Between these two points, traffic will begin diverting from the frontage road back to the main lanes, leaving the frontage road underutilized.

These general statements lead to the following equations. If the main lanes are not broken down, and $q_T < N_M \delta Q_M$ (Region 1 on Figure 2), then

$$q_F = 0$$
$$q_M = q_T - q_F = q_T$$

If $q_T \leq (N_M Q_M + N_F Q_{FB})$ (Region 2), then

$$q_F = N_F Q_{FB} * (q_T - N_M \delta Q_M) \;/\; [N_F Q_{FB} + N_M Q_M (1 - \delta)]$$
$$q_M = q_T - q_F$$

If $q_T > (N_M Q_M + N_F Q_{FB})$, then the main lanes break down.

If the main lanes have broken down, and q_T + average queue $> (N_F Q_F + N_M \gamma Q_B)$ or if $1.5 q_T + s_0 > 1.5 N_F Q_F + N_M Q_B (\gamma + 0.5)$ (Region 3), then

$$q_F = N_F Q_F$$
$$s_1 = s_0 + q_T - N_M Q_B - N_F Q_F$$
$$q_M = q_T - q_F + 0.5(s_0 + s_1)$$

If $1.5 q_T + s_0 > 1.5 N_M Q_B + 1.5 N_F Q_F (1 - \theta) \;/\; (\gamma - \theta)$ (Region 4), then

$$q_F = N_F Q_F * [1.5 q_T + s_0 - N_M Q_B(\theta + 0.5)] \;/\; [1.5\, N_F Q_F + N_M Q_B (\gamma - \theta)]$$

$$s_1 = s_0 + q_T fa - q_F - N_M Q_B$$

$$q_M = q_T - q_F + 0.5(s_0 + s_1)$$

If $1.5 q_T + s_0 < 1.5 N_M Q_B + 1.5 N_F Q_F (1 - \theta) / (\gamma - \theta)$, then the main lanes recover.

SPEED-FLOW RELATIONSHIPS

The following parameters and variables are used:

q_M = flow + average queue on main lanes,
q_F = flow on frontage road,
u_M = speed on main lanes (mph),
u_F = speed on frontage road (mph),
N_M = number of main lanes,
N_F = number of frontage-road lanes,
U_{MF} = main-lane free-flow speed (mph),
U_{MC} = main-lane speed at capacity (mph),
U_B = main-lane and frontage-road speed under forced flow (mph),
U_F = frontage-road free-flow speed (mph),
S_{MA} = slope of main-lane curve in Level-of-Service (LOS) A-D range (miles/vehicle),
S_{ME} = slope of main-lane curve in LOS E range (miles/vehicle),
S_{FA} = slope of frontage-road curve in LOS A-D range (miles/vehicle),
Q_M = main-lane capacity (vphpl),
Q_B = main-lane capacity after breakdown (vphpl),
γQ_B = used after breakdown; main-lane flow + average queue at which speeds start to increase (vphpl),
Q_F = frontage-road capacity (vphpl), and
Q_{FD} = LOS D/E breakpoint for frontage road (vph).

After the flow plus average queue has been obtained for both the main lanes and the frontage road, speed-flow curves are used like those in Figure 3. These curves are easy to use and do not require explanation. The main lanes use different curves for congested and uncongested conditions to allow greater flexibility in adjusting the response of the model.

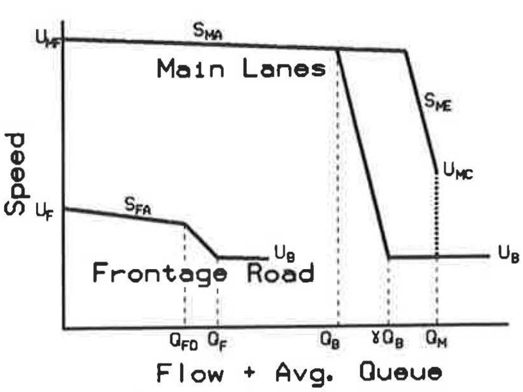

FIGURE 3 Typical speed-flow curves.

SIMULATING EFFECTS OF SC&C SYSTEM

There are five mechanisms in the model that differentiate between the operation of an uncontrolled freeway and a controlled one. The first deals with diversion of traffic to the frontage road before the main-lane breakdown. On a controlled freeway, the surveillance system will detect the freeway approaching congestion and the system will use the ramp metering to divert traffic to the frontage road and help the freeway continue to operate smoothly. The model assumes, on the other hand, that there will be no diversion from an uncontrolled freeway until capacity is reached and the main lanes break down. The model uses δ and Q_{FB} to simulate this effect. δ is a factor applied to the main-lane capacity to indicate the point at which diversion starts. Q_{FB} is the volume on the frontage road when the main lanes break down.

There is also a difference in how traffic is diverted when the freeway is trying to recover. θ is a factor applied to the main-lane capacity after breakdown that indicates the main-lane volume at which all the frontage-road traffic would revert to the main lanes. Because there will generally be some traffic on the frontage road until the main lanes have completely recovered, this factor should be less than 1. A controlled system should, however, retain considerably more vehicles on the frontage road until recovery, and therefore θ should be lower for the controlled freeway.

Because of lessened turbulence at entrance ramps, a controlled freeway should have a larger main-lane capacity than an uncontrolled one. A study in Austin, Texas, showed a 10 percent increase in main-lane capacity by using ramp metering (B. G. Marsden, unpublished data). The model uses Φ as a factor to increase the main-lane capacity to reflect this.

There should also be an increased capacity along the frontage road, because a central signal system will be able to adjust to conditions and provide better service than isolated interchanges. It would also be possible to change the phasing sequence and offsets to enhance progression along the frontage road in cases of diversion leading to a higher free-flow speed as well as to higher capacity.

SENSITIVITY TESTING

Because of the large number of parameters, extensive testing was not possible on all of them. However, several key and questionable ones were chosen for sensitivity testing (Table 1). Those parameters that were tested over a range of values were analyzed at ADTs of 150,000; 160,000; 170,000; 180,000; 190,000; and 200,000 on a freeway with six main lanes and four frontage-road lanes. Most of the parameters that were tested over a range did not significantly affect the results. Three parameters were, however, significant. The most important was the main-lane capacity after breakdown (Q_B):

Q_B	*Hours of Delay Saved*
1,500	20,800
1,600	15,600
1,700	10,700
1,800	7,100

TABLE 1 PARAMETERS CHOSEN FOR SENSITIVITY TESTING

Parameter	Description	Value(s)	Source
Q_M	Main-lane capacity (vphpl)	1,800-2,000	HCM *(4)*
Φ	Factor to increase main-lane capacity for controlled system	1.05-1.15	Summer *(3)*
Q_B	Main-lane capacity after break-down (vphpl)	1,500-1,800	HCM *(4)*
Q_F	Frontage-road capacity (vphpl)	800-900	HCM *(4)*
λ	Factor to increase frontage-road capacity for controlled system	1.25-1.35	
Q_{FB}	Frontage-road volume during main-lane breakdown (vphpl)	500-700	
δ	Factor applied to main-lane capacity to indicate when diversion to frontage road on controlled system begins	0.85-0.95	
γ	Factor applied to main-lane capacity after breakdown to obtain a capacity above which the frontage road is fully loaded	1.1-1.2	
θ	Factor applied to main-lane capacity after breakdown to obtain main-lane volume at which all frontage-road traffic would revert back to main lanes	0.80-0.95	
U_{MF}	Main-lane free-flow speed (mph)	60	Memmott *(1)*
U_{MC}	Main-lane speed at capacity (mph)	35	Memmott *(1)*
U_B	Main-lane and frontage-road speed under forced flow (mph)	15	Memmott *(1)*
U_{FC}	Controlled frontage-road free-flow speed (mph): LOS A for 45-mph arterial	35	HCM *(4)*
U_{FU}	Uncontrolled frontage-road free-flow speed (mph): LOS C for 45-mph arterial	22	HCM *(4)*
S_{MA}	Slope of main-lane speed curve in LOS A-D range	0.002	Memmott *(1)*
S_{ME}	Slope of main-lane speed curve in LOS E range	0.073	Memmott *(1)*
S_{FA}	Slope of frontage-road speed curve in LOS A-D range	0.012	Memmott *(1)*
Q_{FD}	LOS D/E breakpoint for frontage road (vphpl)	600	Memmott *(1)*

Note: HCM = Highway Capacity Manual; LOS = level of service.

The main reason for this is that the capacity after breakdown is very influential in determining the length of time that a queue will be present. Increasing it significantly decreases the hours of delay under an uncontrolled situation.

The increase in main-lane capacity due to ramp meter control (Φ) was also a significant parameter:

Φ	*Hours of Delay Saved*
1.05	18,600
1.10	20,800
1.15	25,700

The main-lane capacity (Q_M) also caused a significant change in the delay savings:

Q_M	*Hours of Delay Saved*
1,800	19,600
1,900	20,800
2,000	24,700

The major changes seen in these parameters are primarily due to isolated effects from using hourly flows. Over a 20-year analysis, these fluctuations would tend to even out.

Figure 4 shows how the recurrent delay savings behaves over a range of ADTs. At 250,000 ADT, the volumes are so large that both the controlled and the uncontrolled system spend most of the day queued up, thereby reducing the savings.

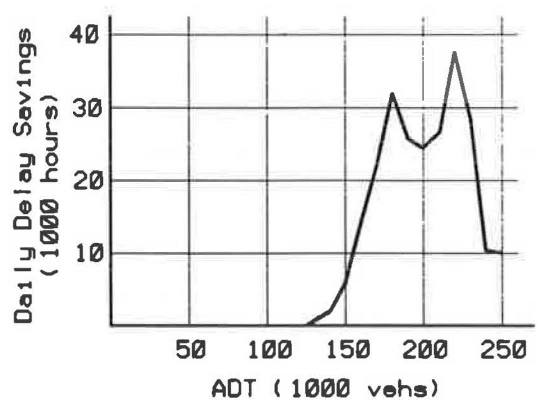

FIGURE 4 Recurrent delay savings over range of ADTs.

NONRECURRENT DELAY

In addition to the recurrent delay calculated by the foregoing procedure, nonrecurrent delay is taken into account. Nonrecurrent delay is most often due to a car that has run out of gas, a flat tire, or a minor accident. The *Freeway Management Handbook* estimates that there are 200 incidents of all types per million vehicle miles on freeways (*3*). The handbook also breaks down these incidents by type and gives the probability of each. Approximately 19 percent of these incidents block only the shoulder and require assistance to clear. Of the total number of incidents, 2.6 percent block one lane and require assistance, whereas only 0.093 percent block two lanes. Because of the low probability that an incident will block more than one lane, only shoulder and one-lane incidents are considered.

The handbook also contains guidelines on the capacity of freeway lanes when an incident is on the shoulder or in one lane. For shoulder incidents, capacity with two freeway main lanes is 3,000; three lanes, 4,600; and four lanes, 6,300. For incidents that block a lane, the capacity for a two-lane roadway is 1,300; three lanes, 2,700; and four lanes, 4,300. The capacity of the frontage road is assumed to be 750 vphpl for uncontrolled freeways and 1,000 vphpl for controlled ones.

In the Houston area, it is estimated that surveillance will shorten the response and clearance time of an incident from 35 to 30 min. That is, for the uncontrolled freeway, the period of blockage is 35 min, after which the queue is flushed out at the saturation flow of 1,850 vphpl for the main lanes and 750 vphpl for the frontage road. The controlled freeway is blocked for 30 min, after which the saturation flow is 1,850 vphpl for the main lanes and 1,000 vphpl for the frontage lanes.

Using the annual ADTs found earlier, a daily nonrecurrent delay savings is found by comparing the delay found on a controlled freeway corridor with that of an uncontrolled corridor. An average hourly flow is found for each 4-hr period of the day by using *K*-factors for a normal Texas urban section. The number of shoulder and one-lane incidents is then found for the hour by multiplying the number of vehicles by the length of the section by 0.0002 (200 incidents per million vehicle miles). The graphical technique shown in Figure 1 gives the delay for each incident.

For those incidents during lightly traveled hours that do not congest, it is also assumed that vehicles will travel 30 mph when there is an incident on the shoulder and 15 mph when it is in one of the lanes.

After the daily nonrecurrent delay has been calculated, it is added to the recurrent delay for a total daily delay, which is factored up to obtain the annual delay.

CONCLUSION

This model is intended as a tool to compare various SC&C projects and to obtain an estimate of their benefits. As such, it appears to perform quite well. Many of the assumptions made do not have solid field validation but represent reasonable values, and the sensitivity analysis indicates that the major parameter to be careful of is the capacity after breakdown. The methodology is adaptable and the parameters can be changed for various conditions, such as the use of a parallel street rather than a frontage road. In the context of a planning analysis, it performs admirably well.

ACKNOWLEDGMENT

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Traffic Detector Errors and Diagnostics

LEON CHEN AND ADOLF D. MAY

The results from research into the use of vehicle detectors, with an emphasis on the diagnosis and correction of detector errors, are described. Of primary interest is the development of a diagnostics scheme in which the average vehicle on time is examined as a test statistic. By comparing this value against the average on times for a station of detectors, the validity of detector operation can be checked. This scheme has been tested at the San Francisco-Oakland Bay Bridge and has been found to yield good results. The false-alarm rate is low compared with that for the occupancy diagnostic method, and sensitivity to true detector failures is improved. This test has also been carried out on inductive loop data from Los Angeles and Chicago. Other experimental work has shown that for magnetometers, the measurement of occupancy is greatly influenced by the manner in which the detector is tuned. Thus methods for improving the consistency of detector tuning and minimizing errors are suggested. It has also been found that pulse breakups are a common operational problem, especially in congested conditions and with heavy vehicles. This can lead to errors in measured occupancy and counts of several percent. Tests have shown that breakups are inherent in the design of the hardware, but that compensation can occur with software. An algorithm for this has been designed and implemented that reduces these errors and improves estimation of vehicle lengths. Missed vehicles, spurious pulses, and lane changes have been found to constitute a small fraction of abnormal detector signals.

Many freeway projects incorporate electronic surveillance equipment in their design. An important device in these installations is the vehicle detector. Detectors can supply fundamental traffic data, such as vehicle flows and occupancies. In addition, detector information can be used to evaluate the operation of a freeway segment by providing measures of system effectiveness.

Detector systems can also take more active roles. Ramp control algorithms frequently use local on-ramp and mainline measurements as input to a metering system. Incident detection systems also use segmentwide measures to automatically signal congested conditions. Detection can also be integrated with ramp control, feeding back information under severe conditions.

The successful implementation of automatic detection and control is dependent on its reliability. Removing the human operator from the control loop allows a computer system to continuously monitor large numbers of detectors over wide areas. A drawback is that incorrect detector information can lead to erroneous signaling of incidents.

In this paper problems related to vehicle detector reliability in surveillance systems and methods for compensating for undesirable behavior are discussed.

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DETECTOR RELIABILITY

Several studies have examined the reliability of detectors on freeways and at signalized intersections and determined empirical rates of failure. Tarnoff and Parsonson, for example, accumulated considerable information from the maintenance records of agencies in different parts of the country. They report failure rates between 0.13 and 0.29 failure/detector-year (*1*).

In a separate study, Dudek obtained empirical information from the Gulf Freeway to calculate a failure rate of 1.18 failures/detector-year. Approximately 100 detectors were studied over the period of 5 months (*2*).

As part of the current research project, a study was conducted using the computerized surveillance in Los Angeles. On a section of freeway with 115 detectors, the performance of the loops was monitored for 4 1/2 hr on each of 2 days. It was found that between 10.5 and 14.8 percent of the detectors were unavailable and that between 1.7 and 11.3 percent showed error flags during the experiment. Because the detector error algorithm used by the California Department of Transportation (Caltrans) occasionally flags on congestion as well as detector failure, these figures may not be correct. Conversely, there could be problems that are not evident from the computerized record and diagnostics. It is clear that a significant proportion of detectors can be out of order at any one time. During discussion with personnel from other areas, these figures were found to be considered normal.

Hale summarized loop failures in a survey of maintenance records from 26 states. According to this report, causes for failure include moisture, loop sealant deterioration, pavement cracking, broken wires, deteriorated insulation, corroded splices, and detuned amplifiers. This agrees with information from an FHWA report (*3*) that lists as causes of loop failure detector unit failure, utility construction, poor sealant, pavement cracking and moving, inadequate electrical connections, and lightning surges.

In addition, Ingram cites detuned amplifiers and loops as a cause of detector lockup (*4*). In his 1979 report, he states that loop inductance changes and amplifier tuning point drift affect the operation of the equipment. These may be due to changes in temperature, which cause thermal expansion. Interestingly enough, Ingram has also investigated the accuracy of the detectors. An average loop may give occupancy errors of 7 to 40 percent. A good system is cited as accurate to approximately 5 percent.

DETECTOR ERROR DIAGNOSTICS

For automatic surveillance systems, incorrect information may be worse than the lack of data. In order to flag the correctness of input signals, a number of methods have been developed by traffic engineers to test detector data. Many of these are described in Table 1.

This summary is the result of an investigation of the literature and a research survey. The latter was a questionnaire completed for 32 major freeway projects in North America. One question in this survey was the ranking of detector diagnostics in surveillance. For 19 projects, it was indicated that diagnostics was a high research priority. Of these, it is noted that 17 were operational, with some form of data acceptability test already in place. Current experience with detectors clearly establishes the need for improved error checking (*5*).

Information was provided by most projects on how they monitor the validity of their data. These are categorized in Table 1 according to the data parameter being examined. It is clear from the maximum and minimum limits shown in the table that most checks are fairly primitive. The ratio of the upper and lower limits of acceptable values for the count tests is often 10:1 or more. The occupancy comparisons also accept a wide range of values. A commonly defined range allows values from 1 to 95 percent.

Rough checks such as these are necessary because the tests do not change with traffic conditions. A few of the algorithms are dynamic. The Maryland test uses historical values. The Caltrans occupancy test compares an individual lane with other detectors at the same station. Thus detectors check against each other, independent of the traffic conditions. This allows the test window of values to be restricted to 4 to 1. A few exceptional systems are those in Ohio or New Jersey, where longitudinal checks can easily be made.

For a few projects the actual pulses are checked for validity. For the Maryland project the rate of short and long pulses coming in is checked to make sure that upper limits are not violated. The Surveillance Control and Driver Information (SCANDI) system validates a count of long pulses. The Chicago system accumulates the count of short pulses, but only as information for the operator, not as part of an automatic diagnostic. Finally, the New York system computes the average on time of the incoming data, which is a useful statistic, but only provides it for the operator, and not as part of an on-line diagnostic.

TABLE 1 MAINLINE DETECTOR CHECKS

	Data Parameter			
State or Project	Counts	Occupancy	Pulses	Other
I-83, Maryland	Upper/lower based on historical 15 min; 15 min < 1; 15 min > UDUL	—	Percent long > UDUL; percent short > UDUL	—
QEW, Canada	5 min < 1; 5 min > 250	30 sec > 95 percent; 5 min < 1 percent; 5 min > 95 percent	—	—
Howard Frankland Bridge	1 min < UDLL; 5 min > UDUL	1 min < UDLL; 1 min > UDUL; 5 min < UDLL; 5 min > UDUL	—	Speed: 5 min > UDUL
Caltrans Districts 4, 7, 11	—	1 min > UDUL; 1 min > twice station avg; 1 min < half station avg	—	No count within allotted time, based on avg flow
Colorado	UDLL UDUL	—	—	—
Chicago, Ill.	—	—	Short pulse count for operator	—
SCANDI System	UDUL	—	Pulse length > UDUL	No count within allotted time
Minnesota	5 min < 20; 5 min > 250	5 min < 3 percent; 5 min > 80 percent	—	—
New Jersey	—	Longitudinal difference 10 percent	—	—
New York	—	—	15 min avg length for operator	—
Ohio	Closely spaced longitudinal difference > 3 percent	Closely spaced longitudinal difference > 3 percent	—	—

Note: UDUL = user-defined upper limit; UDLL = user-defined lower limit; QEW = Queen Elizabeth Way; SCANDI = Surveillance Control and Driver Information system. Dashes indicate data not applicable.

ON-LINE DATA COLLECTION SYSTEM

To study the behavior of detectors under a variety of conditions, the Institute for Transportation Studies (ITS) developed an on-line data collection system with the aid of Caltrans District 4. This allows data to be gathered under experimental control, with the history and adjustment of the detectors known and changeable. Off-line data supplemented the results obtained from the on-line tests.

The surveillance system at the San Francisco-Oakland Bay Bridge (SFOBB) was chosen for this work. The average daily traffic (ADT) for the bridge is approximately 228,000 vehicles/day, with a two-directional peak hourly flow of about 20,500 vehicles/hr (*6*). There are four magnetometer stations located downstream from the metering station that controls westbound traffic. These are named the O, A, B, and C stations. The surveillance system uses a configuration with a single probe in each of five lanes. The O, A, and B stations are approximately two-thirds of a mile from the SFOBB toll plaza. They are closely spaced, with a longitudinal separation of 10 ft.

The on-line data collection system brings the signals from the magnetometers to a microcomputer at ITS. The data are sampled and stored 60 times per second. In addition, single-pulse error checking or diagnostics may be performed according to user-specified parameters.

OFF-LINE DATA SETS

Los Angeles

In order to generalize the results from the SFOBB experiments, two sets of loop data were obtained from the surveillance system in Los Angeles.

In 1974 the Los Angeles system brought in from the field information from individual detector loops sampled at 1/15 sec. The specific data set studied here is from the westbound Santa Monica Freeway from 6:30 to 9:30 a.m. on a weekday. This section of roadway is covered by 128 detectors on the mainline, collector-distributors, and ramps (*7*).

The new data set was recorded from Orange County Route 22 at 2:30 to 7:00 p.m. on April 15 and 16, 1986. This tape contains 1-min summaries of the loop counts and occupancy times for the eastbound and westbound traffic. Because the current Los Angeles surveillance system aggregates data in the field, it is not possible to replicate all the SFOBB analyses.

Chicago

Another source of loop detector data was the surveillance system in Chicago. The Traffic Systems Center (TSC) is able to bring in and record information from a single lane. This allows TSC personnel to record data from a four-lane station, sequentially switching from one lane to the next, at half-hour intervals. Data were taken from Monday, April 14, 12:00 noon, to Thursday, April 18, 5:00 a.m. In all, 63 hr of data was analyzed for this project. The collection site was a four-lane section of the Eisenhower Expressway.

SFOBB TUNING EXPERIMENTS

Several tuning experiments were carried out at the SFOBB in order to examine the behavior of detectors under normal and unusual conditions. There are a variety of reasons why these were important.

First, in examining collected traffic data, it became apparent that the detectors did not always provide comparable information. This was evident when the detector pulse on-time distributions for the A and B stations were compared. The average on times for the two stations varied by 5 to 10 percent for the five pairs of detectors. This can also be seen in the loop data from Chicago, which were taken over a period of 4 days. Figure 1 shows the distribution of on times for the four lanes studied. The average on time for Lane 1 is significantly longer than that for Lane 2. The difference in modal value for the two lanes is 50 percent. Although differences are expected when lateral comparisons are made, these loops clearly register vehicle presence in different fashions. From the Los Angeles data, Figure 2 shows a similar discrepancy between two lanes on a connector to the Santa Monica Freeway.

Second, the consistency of a measurement is important. The SFOBB metering system is configured to begin control at an average occupancy of 11.5 percent. Other projects use occupancy as an important measure of traffic conditions for control surveillance and incident detection systems. It is thus valuable to know the reliability of the equipment and the consistency of its adjustment and measurement.

Third, in examining detector pulses, it was found that several unusual types of signals were being recorded. Pulse breakups appear as a detection dropout during the passage of a single vehicle. "Misses" are indicated by a signal on one detector with no corresponding signal on a nearby detector.

Finally, in the preceding discussion of detector reliability, it was indicated that sensitivity drift is a common detector failure mode. The primary causes of detector failure (*1-4*) often result in degraded performance because of a detuning effect.

Tuning Experiment 1

The Canoga magnetometers used at the SFOBB are adjusted by a specific procedure developed by District 4 personnel. The steps are as follows:

1. Tuning is best carried out under light to medium flow conditions;
2. The detector is placed in the calibration mode;
3. The knob is turned counterclockwise until the indicator light is off;
4. The knob is turned clockwise until the indicator light flashes steadily;
5. The knob is then turned counterclockwise one-quarter of a turn (referred to as a "turn-back" in the following discussion); and
6. The detector is put into the presence mode. It is now tuned.

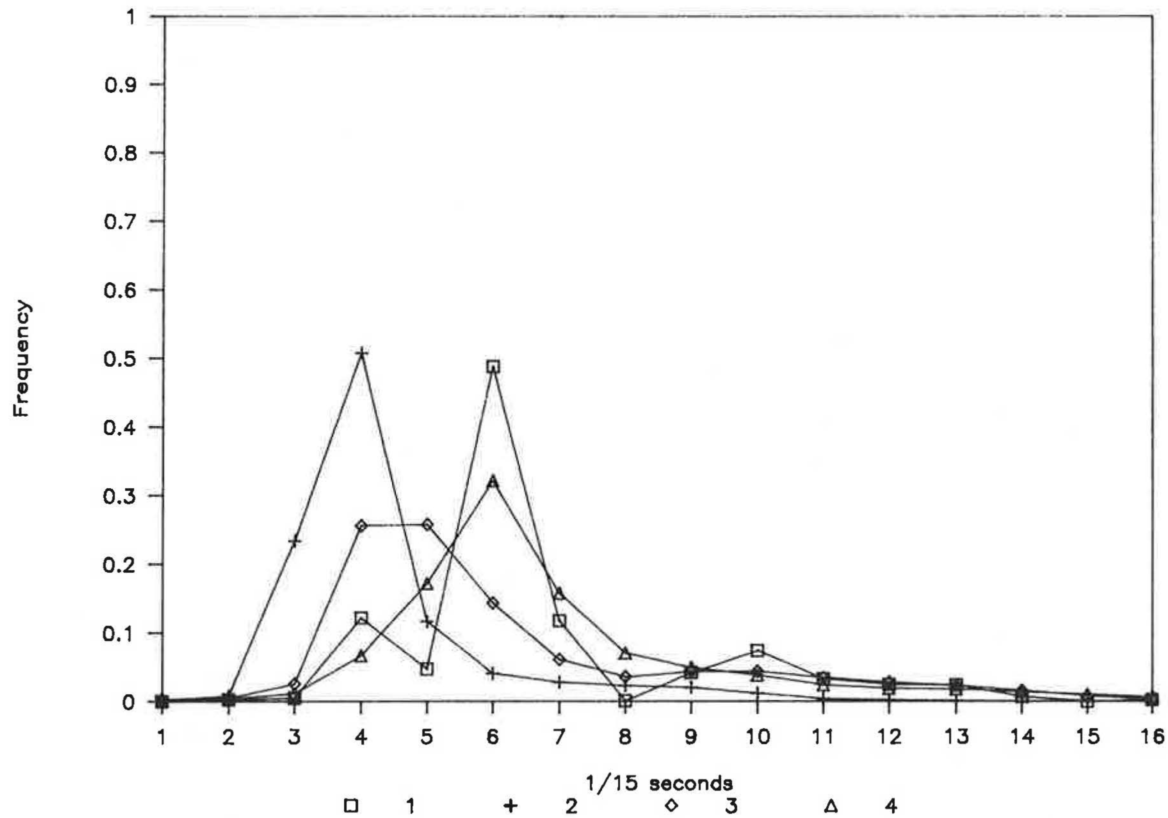

FIGURE 1 On-time distributions, Chicago loop data (April 14, 12:00 noon, to April 18, 5:00 a.m.).

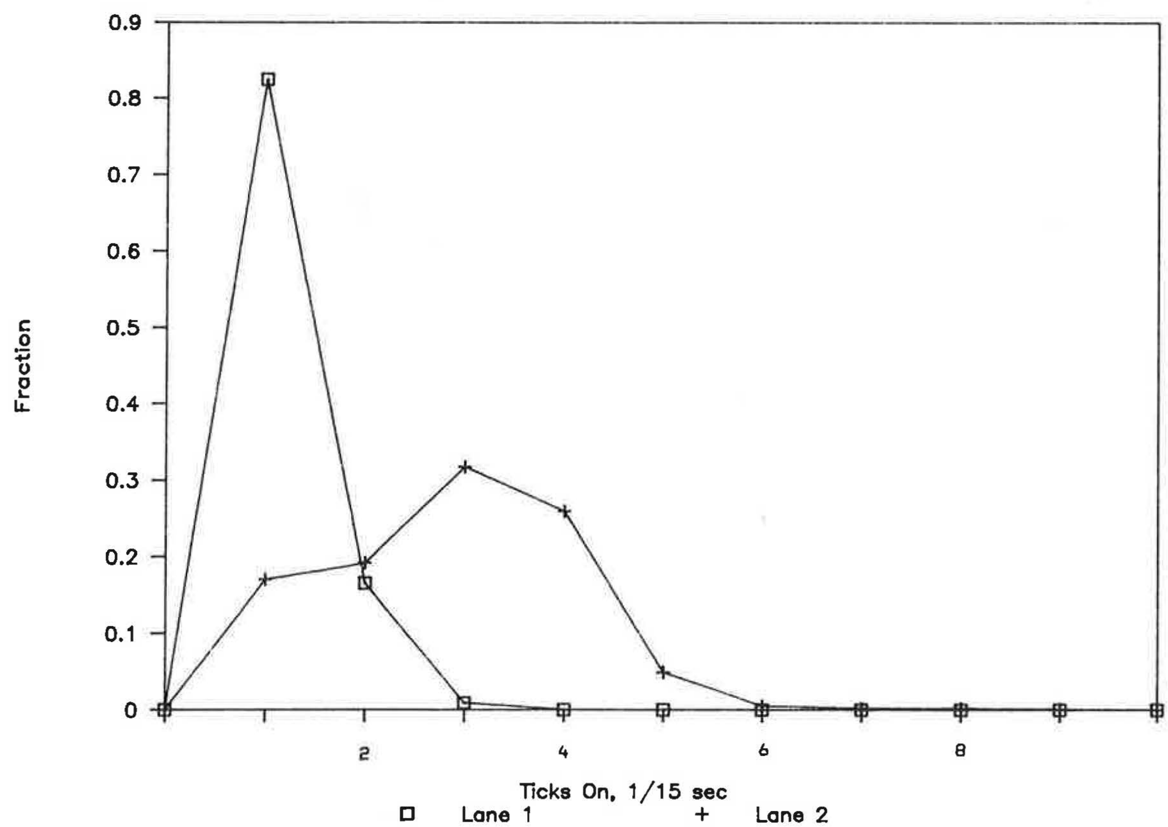

FIGURE 2 On-time distributions, Los Angeles loop data (weekday, 6:30 to 9:30 a.m.).

Two points are important to note at this time. First, the "detuning" in Step 5 has been added by SFOBB personnel. Second, the adjustment knob, supplied by the manufacturer, is normally a one-turn potentiometer. All SFOBB magnetometers have replaced this by a 10-turn potentiometer. This is done because the original design is considered exceptionally difficult to tune.

The first experiment took place on two afternoons with moderate traffic flows. After the A and B stations had been adjusted according to the SFOBB procedure, the detectors of Station A were "turned back" by specified amounts. Station B was always held constant. The data collection system was used to record occupancies for the two stations at 1-min intervals for 2 to 8 min. This procedure was carried out for six different degrees of turn-back, from zero to three-fourths turn.

Results

The data from the first tuning experiment are shown in Figure 3. The horizontal axis gives the turn-back setting of detector Station A. Thus the data at zero are taken with Station A at the manufacturer's tuning and B, as always, is at the Caltrans standard tuning. The vertical axis shows the ratio of the occupancy measurements from Station A versus that from Station B.

The graph thus shows how tuning of a detector affects occupancy values. Near the typical operating point of the detectors, a change of one-eighth turn causes a 10 percent change in measured occupancy. A polynomial least-squares fit is shown. The upward curvature of the fit provides a good explanation for the turn-back procedure used by the SFOBB personnel. Near the manufacturer's tuning, at zero turn-back, an error in adjustment creates a large variation in occupancy. This sensitivity is quantified by the slope of the fitted curve at zero, which is 1.31. At the SFOBB setting, an error in adjustment is still penalized by a large error in the occupancy, but the slope of 1.01 is less.

Conclusions and Recommendations

Experiment 1 clearly shows that the tuning process is central to obtaining comparable occupancy values. A slight misadjustment of a detector amplifier can account for significant differences in occupancy readings between detectors. This is noteworthy because the Caltrans amplifiers, with 10-turn potentiometers, are easier to tune.

Immediately following tuning, the between-station discrepancies are reduced to a few percent. Because occupancy is an important measure for operations, it is advantageous to minimize any source of discrepancy. There are several recommendations to help the tuning process:

1. The judgment of "steadily flashing" can be problematic, especially for inexperienced personnel and in heavy flow conditions, because vehicle triggerings cause flashing. If possible, flashing should be calibrated to a standard frequency in minimal traffic.

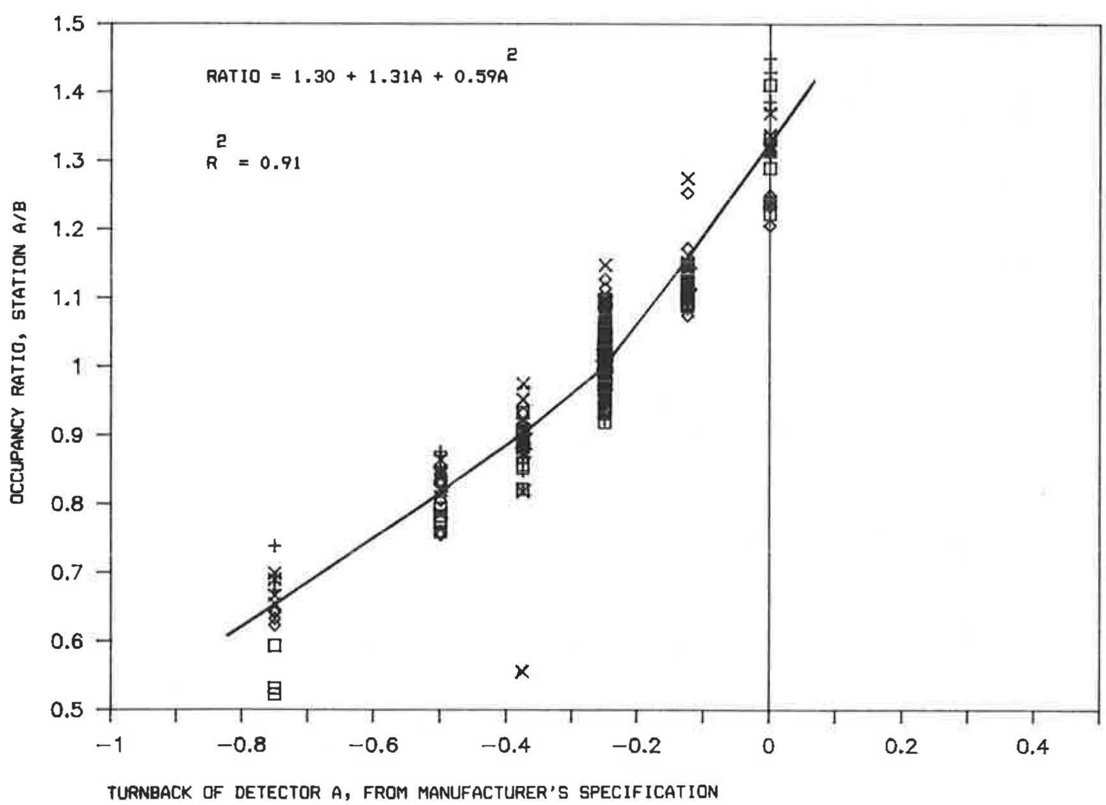

FIGURE 3 Tuning experiment results.

2. It is helpful to modify the original equipment to facilitate the tuning process. The replacement of the standard potentiometer is an example. Additional resistive circuitry could be added for more sensitivity.

3. The turn-back is important and should be standardized. A dial could be added to the knob to make the turn-back more consistent.

Tuning Experiment 2

The second tuning experiment at the SFOBB was carried out to see if pulse breakups were due to tuning. Because they were occurring regularly, it was postulated that the reduced sensitivity of the SFOBB detectors, due to the tuning procedure, might be responsible for these failures.

Procedure

The effect of tuning on pulse breakups was examined in two data sets of 2 hr each. The first set was taken with one-half turn-back on Station A while Station B was held at one-fourth turn-back. The second data set was taken with zero turn-back on Station A while Station B remained at one-fourth turn-back.

The gap-time distribution of each data set was then examined. Other experimental work, described later, indicates that short gap times are clear indicators of pulse breakups. The count of gap times less than one-fourth of a second is then a count of pulse breakups.

Results and Conclusions

It was found that there are no significant differences in the frequency of short gap times, regardless of whether the detector is at the manufacturer's specification or the SFOBB setting of lessened sensitivity. Thus breakups are inherent in the use of magnetometers. The hardware deficiency may be correctable with software or a different probe configuration with more sensors.

SFOBB VIDEO SURVEILLANCE EXPERIMENT

The video surveillance experiment at the SFOBB permitted a comparison of the recorded detector data with a visual record of the traffic. As indicated earlier, unusual detector signals were found during close examination of the pulses. In addition to breakups and misses, the sources of short and long on times as well as short gap times were of interest.

Procedure

Three mechanisms were used for observing the traffic near the SFOBB detectors. A video camera recording provided the basic evidence for each vehicle passage. The computerized data collection system provided the detector's indication of vehicular occupancy. Finally, an observer noted and recorded the traffic conditions at the site. These will be described in more detail later.

The video was filmed from a lift truck located on a frontage road adjacent to the A and B detector stations. While the filming took place, the data collection system recorded the detector information every 1/60 sec. To maintain synchronization between the data collection clock and the video clock, a circuit was designed to give a simultaneous pulse in the computer record and a visible indicator in the film.

The Data Set

The video experiment took place from 10:00 a.m. to 12:00 noon in clear, dry weather. On this day there were two incidents, one major and one minor. This gave approximately 20 min of congestion data; during the remaining time there was free flow at a volume of approximately 8,000 vehicles/hr.

Data Analysis

The experimental data were analyzed in several steps. First, unusual detector data were selected from computerized records for extended investigation. Second, the selected computer data were matched with the video film to log vehicle movement and type, and the detector behavior was checked for correctness. Finally, incorrect detector behavior was cross-tabulated with vehicle movement and type.

Suspicious detector data were extracted from the computer data set by examining the individual signals and by comparing the pulses at the upstream and downstream detectors. For the individual pulse check the following criteria were used:

1. Short on times (less than 1/12 sec),
2. Long on times (longer than 1/2 sec),
3. Short gap times (less than 1/4 sec), and
4. Long gap times (longer than 1 min).

The short on time is equivalent to the Chicago surveillance system loop detector criterion adjusted to the smaller detection zone of magnetometers. The long on-time value selects approximately 2 percent of the pulses from a newly tuned SFOBB detector under free-flow conditions. The long gap-time figure is based on the existing Caltrans detector lockup tests for heavy flow conditions. The short gap time picks out an apparent mode that was seen at the low end of the experimental off-time distribution.

Comparisons also are made between the signals recorded from the A and the B stations. If there is an apparent miss or breakup on one of the stations, it is marked for further examination.

If a pulse is selected as suspicious, the preceding and following pulses for the two longitudinal detectors are written into a file for the next step, which compares the pulse with the video record.

To compare the two sources of information, software was developed that graphically displays the pulse sets in conjunction with the estimated time of vehicle passage according to the video clock. This allowed the observer to match virtually all vehicles with their accompanying pulse. In the comparison, it is possible to deduce detector behavior and vehicle type and movement. The information on the detector performance and vehicle information are cross-tabulated in a database. Lanes 1, 2, 3, and 5 were examined. Lane 4 was omitted, because only one station was functional, which precluded the paired-detector-data screening step.

Results

A primary finding of this experiment is that pulse breakups are an important mode of detector failure. This applies to both passenger cars and trucks in both congested and free-flow traffic. Table 2 summarizes the data for the four lanes studied. These breakups were all selected from the data set by the short-gap-time criterion. Overall, a short gap was found to be a reliable predictor of breakup, correctly flagging signal dropout 94.6 percent of the time under both free-flow and congested conditions.

TABLE 2 SHORT GAPS DUE TO BREAKUP

Condition and Lane	Percentage by Vehicle Type and Station			
	Passenger Cars		Trucks and Buses	
	A	B	A	B
Congested				
1	20.5	10.9	NA	NA
2	14.5	12.8	10.0	10.0
3	10.5	9.4	10.0	27.0
5	9.4	11.6	32.0	46.0
Free flow				
1	2.0	0.84	30.0	25.0
2	1.2	1.1	26.5	29.0
3	1.7	2.3	29.6	32.0
5	0.96	0.5	49.0	47.0

It is evident that congestion causes a higher rate of breakup errors for passenger cars in all lanes. It is also suspected that this is the case for trucks, but this is not obvious from the statistics. It is believed that the short-gap-time criterion fails to diagnose breakups with the slower speeds because the dropout times begin to exceed 1/4 sec. The short-gap-time criterion also does not indicate the triple or quadruple triggerings that occur occasionally. These are regularly caused by the passage of a twin trailer truck.

A second interesting finding is that unusual vehicle movements account for few of the unusual detector pulses. An example of this would be a lane change over the detector stations, which might give a short on time or a pulse breakup. Occasionally a short gap time was the result of close vehicle headways. Suspicious detector pulses were rarely caused by vehicle movement.

A third finding is that few pulses are due to adjacent-lane triggering or are spurious signals. Of the 3,061 pulses examined, only 18 were due to a truck in an adjacent lane when no vehicle was in the lane for which the pulse was triggered. Only 10 pulses could not be accounted for by the vehicle. These were all less than 5/60 sec long.

Finally, motorcycles do not appear to account for many of the short pulses being recorded. This is because there are less of them in proportion to other vehicle types, and it appears that they are often not registered at all by the detector. This is probably due to their small size and the fact that motorcycles generally drive away from the lane center.

Conclusions and Recommendations

The primary finding of the surveillance experiment is that pulse breakups are prevalent and thus require compensation in software. As indicated in the tuning-experiment discussion, breakups do not appear to be caused by incorrect hardware adjustment. They may be due to probe number and layout. In District 4, correction is accomplished by a specific counting algorithm. In order to register a valid vehicle count, a minimum gap time is required, followed by minimum on time. For different installations, the gap time requirement is 0.2 to 0.5 sec, and the on-time requirement is 0.07 to 0.1 sec.

SFOBB calculations include all detector on time in the occupancy accumulation, but a count does not occur until the foregoing conditions are satisfied. This generally works, but has drawbacks that are important for detector error diagnostics, described later. First, the dropout time is not accumulated in the occupancy figure. Second, vehicle length is not accurately recorded, because the on-time count ceases as soon as the detector turns off.

A counting procedure can be used to correct these problems. Short gaps can be interpreted as dropouts from a pulse breakup. Because of this, the detector is altered to the on state for the gap. Short pulses that are not part of a breakup are converted to the off state, and are effectively ignored. These rules use the finding that most short gaps are the result of a breakup and that short pulses are usually part of a breakup or a spurious signal. The consequence of this new procedure is to correct the occupancy and count calculations for breakups. Error under different scenarios without this compensation is shown in Table 3. The numerical differences are not large when compared with District 4 methodology, but an important result is the generation of a correct pulse length for later analysis. This is important in vehicle identification and detector diagnostics.

ON-TIME DIAGNOSTIC TEST

As discussed earlier in this paper, Caltrans has several tests to examine the functioning of mainline detectors. The lockup test flags an error if a detector fails to change state in a designated amount of time. The occupancy test looks for a detector that reads significantly higher or lower than other detectors at the same station.

Several results from the preceding experiments are important in the development of a more advanced diagnostic scheme. As such, they bear repeating:

TABLE 3 MEASURED ERROR FROM PULSE BREAKUPS WITH DIFFERENT COMPENSATION TECHNIQUES

	Percentage by Type of Traffic			
	Typical Mix		Rightmost Lane	
	Free Flow	Congested	Free Flow	Congested
No compensation				
Count error	+1.97	+12.7	+5.8	+14.0
Occupancy error	-0.4	-1.6	-0.85	-1.6
Caltrans District 4 compensation				
Count error	0.0	0.0	0.0	0.0
Occupancy error	-0.4	-1.6	-0.85	-1.6

Note: Assumptions for calculations, based on SFOBB experimental data, are as follows: Typical mix is 98 percent passenger cars, 2 percent trucks and buses. Rightmost lane is 86 percent passenger cars, 14 percent trucks and buses. Average pulse length, 12/60 sec for passenger vehicles, 25/60 sec for trucks and buses. Average breakup gap, 3/60 sec. Congestion speed, 30 mph; breakup gap, 3/60 sec.

Percentage of breakups is as follows: passenger cars—free flow, 1.33 percent; congested flow, 12.5 percent; trucks—free flow, 33.5 percent; congested flow, 24.0 percent.

1. Variations in sensitivity and tuning account for shifts in the distribution of on times. This variation can be quite large.
2. Pulse breakups can be identified and corrected by an algorithm that modifies short gaps and pulses. This yields correct pulse lengths and allows identification of long vehicles.
3. The on-time distribution appears quite similar to a normal distribution, although the normal is slightly less peaked in the center.

The Caltrans occupancy test often fails to pick up shifts in sensitivity because of the wide error margins, which allow for normal variations in occupancy. The average on time appears to be a good measure when compared with occupancy, because occupancy directly varies with flow rate. On time per vehicle eliminates this variability.

Occupancy also increases when trucks and buses are in the vehicle mix. By filtering out long vehicles from the on-time average test statistic, the resulting variance can also be reduced. This makes compensation for truck pulse breakups important in the data-processing procedure.

In general, a particular lane will yield higher or lower average on-time values on the basis of the speed distributions and amplifier tuning. This can be eliminated by using a historical factor that accounts for these long-term differences. This allows direct comparisons to be made between lanes.

Finally, under heavy congestion and incident conditions, there can be large short-term fluctuations in any microscopic traffic characteristic. It is therefore desirable to flag the detector diagnostic as questionable in those situations. A simple test for congestion is the average speed at the station. This can be estimated from the station volume and occupancy.

In sum, this procedure has similarities to the Caltrans occupancy check, but has many extensions. It should be noted that vehicle speeds change the on time. The algorithm compensates by comparing against a station average, which reflects aggregate vehicle speeds. Thus, a lane speed bias will generate a false alarm only if it is marked and not compensated for by the historical lane factor.

Statistically, the algorithm is similar to a two-sample problem in which a sample from one lane is compared with samples from other lanes, as represented by the station average. This is appropriate, because the on time is distributed as a normal random variable. The test determines whether the detector in one lane is behaving significantly differently from those in others.

A convenient sampling interval is 5 min. Under moderate traffic conditions, this gives a lane sample of 50. With a typical on time of 12/60 sec, the diagnostic flags an error if the sample differs from the station mean by approximately 15 percent. The designed test will signal if a lane is greater than 115 percent or less than 85 percent of the station on-time average.

SFOBB Experiments

In order to evaluate the described on-time algorithm, several blind tests and an extended implementation were run. For the blind tests, on two mornings arrangements were made for Caltrans engineers to alter the tuning of an arbitrary detector by one-fourth turn while the data collection system was running. The on-time algorithm was then used to pick out the simulated failure. The results of the diagnostic were then checked with SFOBB personnel.

SFOBB Results

During the 2 1/2 hr of the first test, the traffic flows were heavy, but not congested. Figure 4 shows the test statistics derived from the on-time ratio diagnostic algorithm. The results clearly show the time and lane of the detector failure without ambiguity.

By contrast, Figure 5 shows an occupancy ratio test applied to this same data set. Given a time-series view of the data, it is possible to see the abrupt "failure" of Lane 5. But at any given point in time, it would not be possible to distinguish it from the remaining lanes. In fact the "failed" detector measures

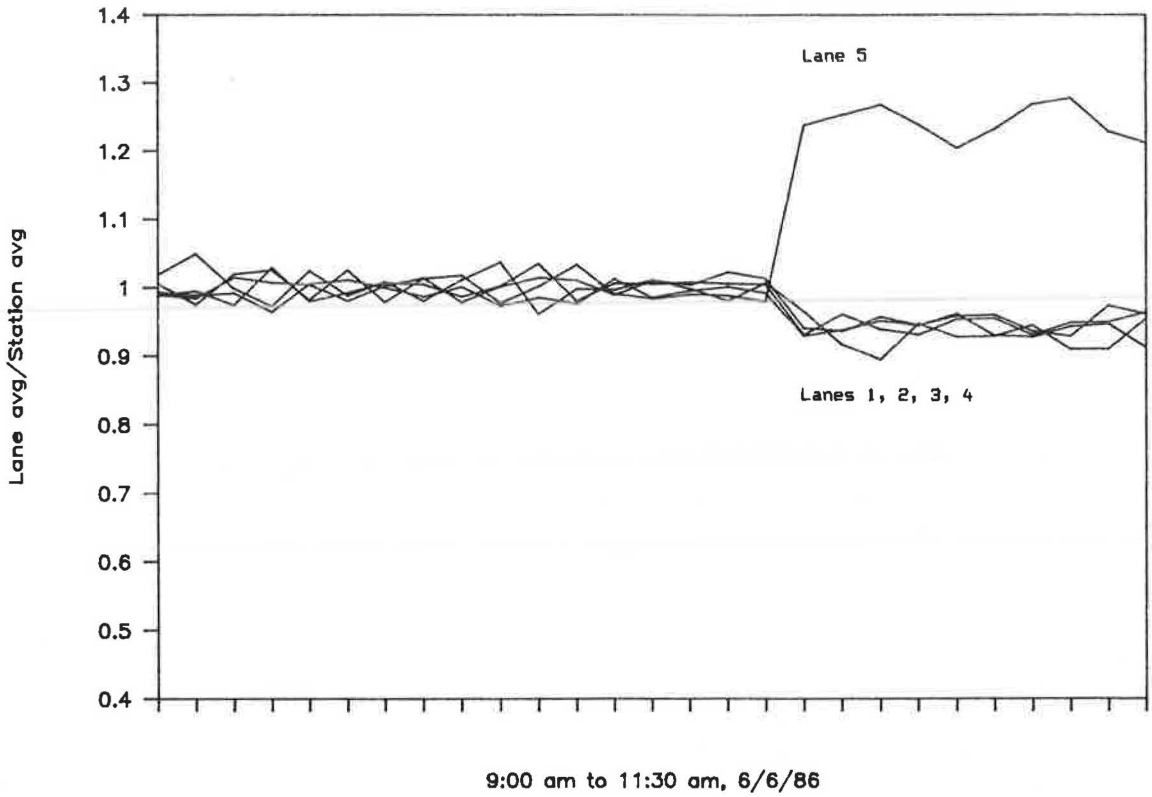

FIGURE 4 On-time diagnostic test: lane versus station average (2.5 hr, 9:00 to 11:30 a.m., June 6, 1986, 5-min averages).

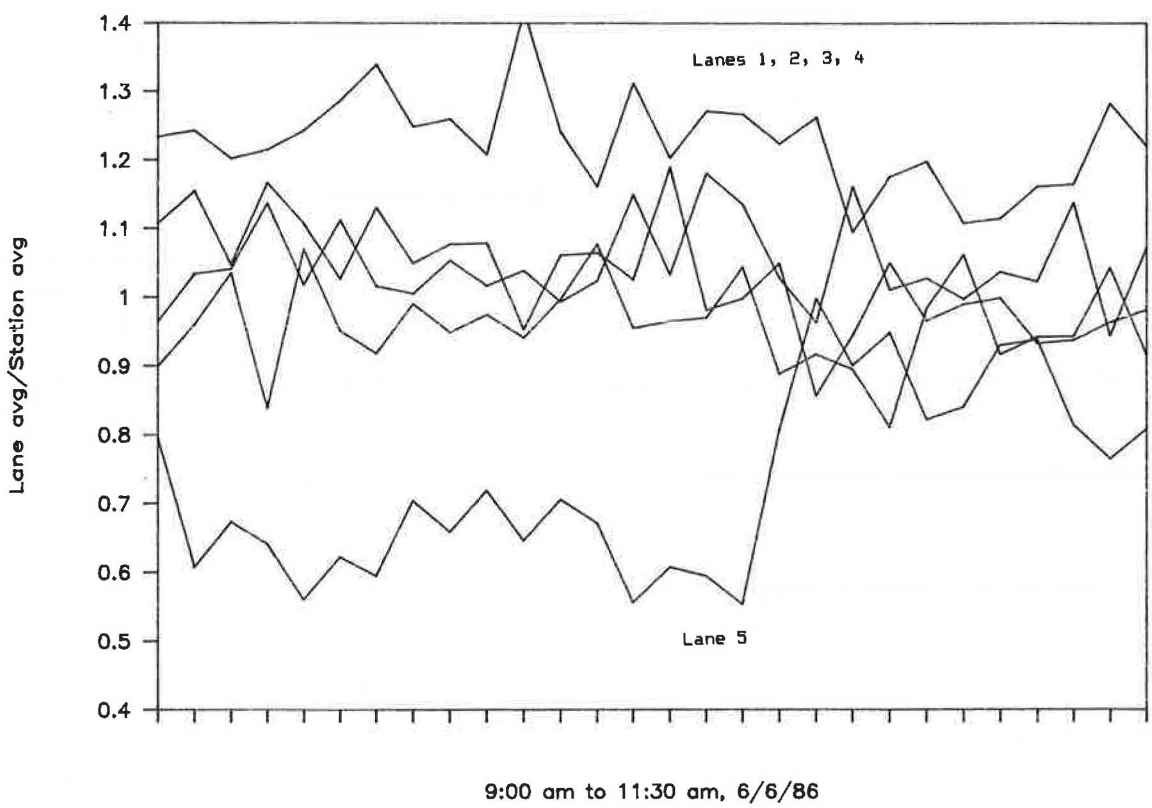

FIGURE 5 Occupancy diagnostic test: lane versus station average (2.5 hr, 9:00 to 11:30 a.m., June 6, 1986, 5-min averages).

occupancies close to those of the other lanes. In addition, the occupancy test presented here was derived with a 5-min average, not the Caltrans 1-min average. This larger sample period always serves to reduce statistical noise, and is thus a conservative modification to this comparison. This change was made to facilitate the programming of the two tests.

The SFOBB implementation of the occupancy ratio test uses the Caltrans criterion of 50 or 150 percent variation for Lanes 1 to 4. For Lane 5, however, they have been forced to extend the range to 25 and 175 percent because the occupancy varies more with heavy truck and bus traffic. There is almost a false alarm in Lane 2, and it is also clear that Lane 5 never fails the occupancy ratio test.

During the second blind test of the on-time ratio algorithm, the SFOBB personnel were not able to get out in the field to alter any of the detectors, as had been planned. The data, however, were unknowingly recorded and processed by ITS. As a consequence, the results were examined with the expectation of a simulated detector failure. It was clear that no detector had "failed," and this assessment was verified by the District 4 engineers.

The extended run of the diagnostic scheme involved a longer-term implementation of the on-time and occupancy ratio tests to compare performance under a wide range of flow conditions. A total of 94 hr of comparison were carried out. As with the former test, the occupancy diagnostic was averaged over 5 min rather than the usual 1 min, a conservative modification.

Given this, the occupancy and on-time tests were examined, and summary statistics are shown in Table 4. Comparing the number of flags for each lane, it is very clear that the on-time test yields a much smaller number of false alarms; it is known that the magnetometers are in good working order. For Lanes 2, 3, 4, and 5, the on-time diagnostic gives only 11 percent of the flags when compared with the occupancy test.

Of additional interest is the last row in Table 4, which gives the number of on-time flags that would have occurred if low vehicle count and congestion conditions were not excluded in the suggested algorithm.

There are clearly an unusual number of potential flags in Lane 1 that were not counted because of the count check. This lane was occasionally closed for maintenance work during the test. The lane closures show that it is important to perform a count check before running any diagnostics. This prevents a large number of false alarms when the detector is in fact working. This also shows that the on-time test can run correctly with three- and four-lane stations.

Off-Line Tests

The on-time average for loop data was also checked by using several of the off-line data sets. With the Garden Grove Freeway data, the occupancy test was run with 1-min averages and the on-time test with 5-min averages. Figures 6 and 7 show the occupancy and on-time statistics for the Bristol station, which has four detectors. There is a large amount of variation in these figures because of congestion during the 4 1/2-hr period. The occupancy statistic shows values outside the 50 to 150 percent range, whereas the on-time statistic does not extend beyond 85 and 115 percent.

Figure 2 shows the on-time distributions for the 1974 Los Angeles data. This is 3 hr of data from a two-lane connector entering the Santa Monica Freeway. Each lane differs from the station on-time average by more than 50 percent, showing that one of the two detectors has probably failed. The difference in behavior between the lanes is also enough to trigger an occupancy test flag.

Conclusions

The two SFOBB blind trials show that the on-time ratio test provides a reliable indication of detector status. Because the on times for average-length vehicles are used for the test sample, there is a minimum of noise obscuring important information about the detector status. For loop detectors, the equipment is susceptible to problems with wire insulation, splices, and installation. Magnetometers, as recorded data sets show, easily drift from their desired adjustment over time. Thus it is important for an algorithm to respond to sensitivity changes.

The long-term test for false alarms and results from the off-line data sets show that the on-time average gives a statistic that is more robust under varied traffic conditions. This would give a more reliable indicator of detector failure to surveillance systems implementing incident detection or control.

FURTHER RESEARCH

In order to extend the applicability of the current research, additional experimentation will be carried out to help generalize to other facilities. At this time, field work similar to that just described is being carried out at a set of 16 loop detectors in Pleasanton, California.

TABLE 4 FALSE ALARMS FOR ON-TIME AND OCCUPANCY TESTS

	Station A					Station B				
	1	2	3	4	5	1	2	3	4	5
No. of occupancy flags	342	235	151	—	110	426	64	4	24	67
No. of on-time flags	5	5	4	—	1	5	2	5	6	50
No. of flags without count or speed check	158	7	7	—	80	170	9	7	5	15

Note: Dash indicates data unavailable because of detector failure.

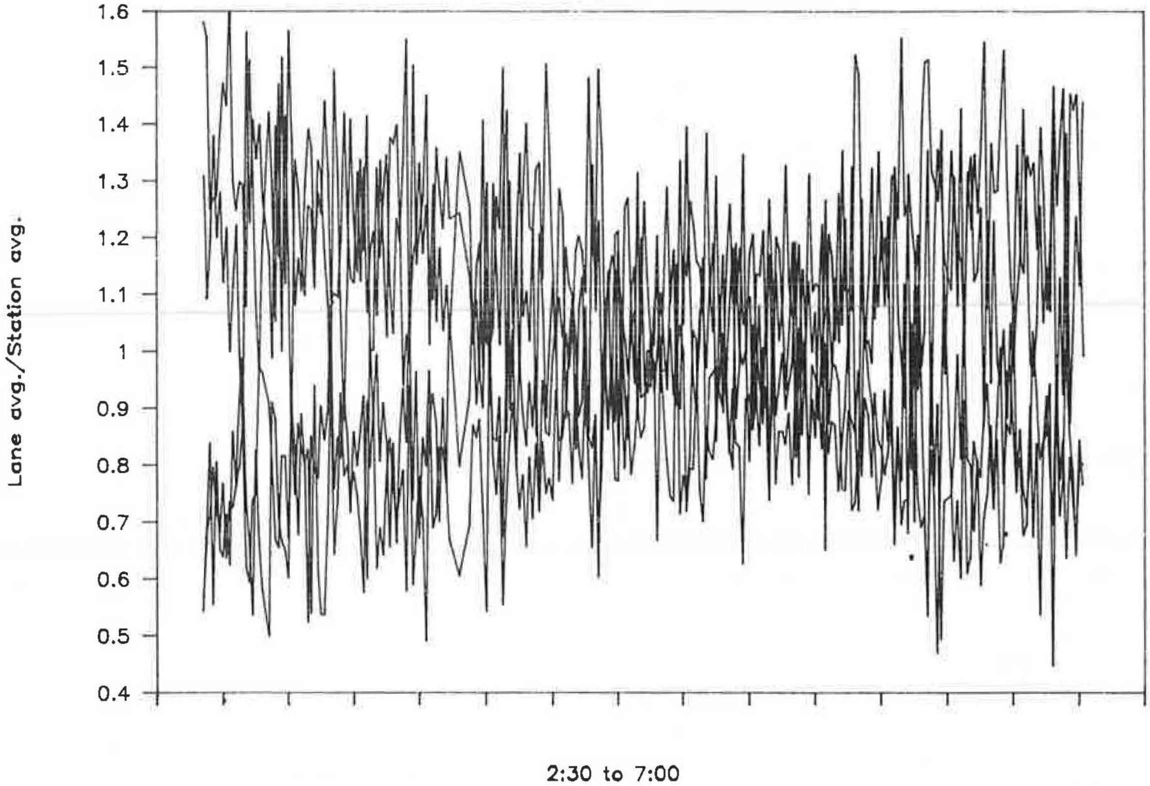

FIGURE 6 Occupancy test for Bristol station: lane versus station average (Orange County Route 22, eastbound, April 16, 1986, 2:30 to 7:00 p.m.).

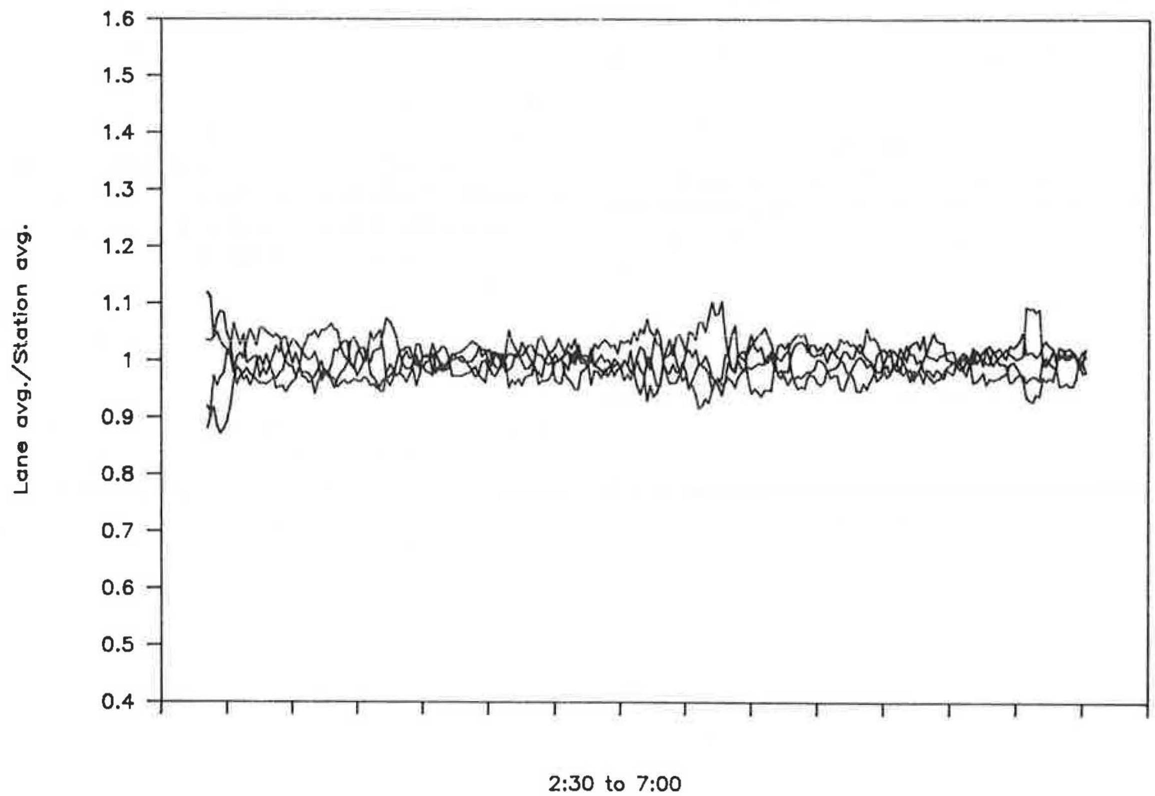

FIGURE 7 On-time test for Bristol station: lane versus station average (Orange County Route 22, eastbound, April 16, 1986, 2:30 to 7:00 p.m.).

CONCLUSIONS AND SUMMARY

A number of important tests examined the behavior of detectors under a variety of traffic conditions. From the study of on-line magnetometers and loops from off-line sources, it is clear that detectors can give misleading information. Some findings have illustrated ways in which basic data can be incorrect. But means have been developed to diagnose and compensate for these errors. First, closely spaced longitudinal detectors at the SFOBB can give occupancy measurements that vary significantly. The same phenomenon is seen when comparisons of data from adjacent loop detectors are made. Results from turning experiments at the SFOBB indicate that this can be due to small changes in tuning and detector sensitivity. Because of this, recommendations for detector tuning and modification are made in this paper.

Second, another form of inaccuracy has been verified by videotaping experiments. Pulse breakups are confirmed to occur with magnetometers at rates between 2 and 33 percent. The highest rates are with trucks and buses, and in congested traffic. The breakups give incorrect measurement of vehicle length, counts, and occupancy unless compensating software is used. A method for doing this is presented and has been implemented on line. Additional tuning experiments have shown that breakups are inherent in the detector design. Examination of inductive loop data indicates evidence of similar behavior.

Finally, a new diagnostic algorithm has been tested that checks the on time per vehicle against a station average. Experiments show good accuracy in flagging changes in detector sensitivity, but the occupancy test does not. An extended run over 94 hr also showed fewer false alarms than the occupancy test, indicating that the on-time ratio is a more robust diagnostic. This is verified with experimental data from Santa Monica and Garden Grove.

By improving the manner in which the basic traffic data are examined, the overall performance of a control, incident detection, or system evaluation scheme can be ignored.

ACKNOWLEDGMENTS

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Freeway Simulation Models Revisited

ADOLF D. MAY

The purpose of this paper is to update and assess the continued development and application of freeway simulation models in the 1980s. Several activities were undertaken to meet this objective. First, literature searches were undertaken utilizing the University of California Institute for Transportation Studies library and the author's personal library. Second, the identified references were classified by freeway simulation model family and placed in a historical perspective. The references were then carefully studied to identify and assess new developments and applications. Finally, identified authors were contacted to determine omissions and to confirm the current status of their freeway-modeling efforts.

The Conference on Traffic Simulation Models was conducted by the Transportation Research Board in Williamsburg, Virginia, June 1981. The conference was sponsored by the Federal Highway Administration and the conference proceedings were published by the Transportation Research Board as *Special Report 194: The Application of Traffic Simulation Models*. The author's paper presented at that conference, Models for Freeway Corridor Analysis, was published in Special Report 194. The paper had two major themes: (a) to describe existing traffic simulation models and their applications in freeway corridor analysis and (b) to demonstrate the need for integration of research, education, and implementation activities as a key to the enhancement of simulation modeling practice. After a brief review of earlier models for freeway corridor analysis, five families of currently available models were described. Particular emphasis was given to the historical development of the models and to real-life applications. The five families of models were

- CORQ
- FREQ
- INTRAS
- MACK
- SCOT

The paper also included an extensive bibliography, which was an attempt to include all published papers in which the development and application of available freeway corridor models were described.

UPDATING PROCESS

The purpose of the current paper is to update and assess the continued development and application of freeway simulation models in the 1980s. Several activities were undertaken to meet this objective. First, literature searches were undertaken utilizing the University of California Institute for Transportation Studies (ITS) library and the author's personal library. Second, the identified references were classified by freeway simulation model family and placed in a historical perspective. The references were then carefully studied to identify and assess new developments and applications. Finally, identified authors were contacted to determine omissions and to confirm the current status of their freeway-modeling efforts. The authors of the various freeway models were most cooperative in identifying omissions and confirming the current status of development, and their responses have significantly aided in completing this updating and assessment process.

The remaining portions of this paper are organized by freeway model family, which is complemented by an extensive list of references.

THE CORQ-CORCON MODEL FAMILY

CORQ and CORCON are the two freeway simulation models in this family. CORCON has been applied to four freeway sites in the Toronto area by the Ontario Ministry of Transportation and Communications since 1978 (*1*). A CORCON6F user's manual has been prepared (*2*).

Yagar prepared a paper in 1980 addressing the question of origin-destination (O-D) demand data requirements for CORQ, because a key ingredient of this model is the assignment (or reassignment) between the freeway and alternative routes in the corridor (*3*). For the past 2 years, Yagar has been making significant improvements in CORQ with sponsorship by the Ontario Ministry of Transportation and Communications. Primary attention has been given to "look ahead" features, computer efficiency, and user-friendliness (*4*). CORQ has not been applied in practice since about 1980. It is anticipated that this model will be a proprietary one.

CORQ has been extensively compared with other models and was selected as the leading one for traffic networks (*5-7*).

THE FREQ MODEL FAMILY

Development and application of the FREQ model family continue during the 1980s at a fairly significant level of activity. These activities will be described in the following paragraphs under priority-entry model development, priority-lane model development, training and technical assistance programs, and model application.

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FREQ7PE was a revised and extended version of FREQ6PE (*8*). Modifications included improved input and output flexibility, fuel and emission options, user-supplied metering plans, queue-length limits, and improvement to traveler-response modeling.

On the basis of extensive training and technical assistance programs, which will be described later, users suggested a number of further improvements for FREQ7PE. The improvements were incorporated into FREQ8PE (*9*). One of the major improvements was the incorporation of the synthetic O-D formulation within FREQ8PE so that users would have the option of directly entering O-D information or entering ramp counts and having the model generate synthetic O-D information.

Two current developments with the priority-entry model family that have not been documented are FREQ9PE and the microcomputer version of FREQ8PE. FREQ9PE is capable of analyzing a 50- to 100-mi length of freeway in one computer pass. In essence, the maximum number of subsections has been increased from 40 to 160, whereas the maximum number of entrances and exits has been increased from 20 to 80 each. The microcomputer version of FREQ8PE is in the final testing stage and release is expected in early 1987.

Turning to the further development of priority-lane models, FREQ6PL was modified and called FREQ8PL (FREQ7PL was never publicly released) (*10*). FREQ8PL became the sister model of FREQ8PE and incorporated many of the features of the latter model that were appropriate for priority-lane investigations, including the synthetic O-D formulation. Currently there is no further development on FREQ8PL, but in the future an increased size version (FREQ9PL) and a microcomputer version are envisioned.

The need for training and technical assistance in the application of simulation models was one of the major conclusions of the June 1981 Williamsburg conference. On the basis of earlier training activities from 1975 to 1981 with FHWA and the California Department of Transportation, and discussion and workshops at the Williamsburg conference, a comprehensive training and technical assistance program was undertaken with the Texas Department of Highways and Public Transportation. Two series of programs were undertaken, one in 1981-1982 (*11-13*) and the second in 1983 (*14-16*). For example, the second series in 1983 consisted of five major stages. In April Workshop 1 was held, which emphasized freeway simulation and calibration (*14*). From April until July, the professionals returned to their urban districts, and with technical assistance, collected input data, made FREQ7 simulation runs, and calibrated model parameters. In July Workshops 2 and 3 were held, with emphasis on ramp-metering simulation using FREQ7PE and priority-lane simulation using FREQ7PL (*15, 16*). From July to December, the professionals, with technical assistance, used FREQ7PE or FREQ7PL, or both, to investigate freeway problems in their districts and possible solutions through entry control and priority lanes. In December the final workshop was held in which the professionals shared their experiences with regard to freeway problem model application and results. Discussions are currently under way with several state departments of transportation, and training and technical assistance programs will probably be undertaken in 1987.

There have been a number of reported applications of the FREQ models during the early 1980s. They are as follows (published reports are indicated by reference number):

- *Impact of Ramp Metering on Carbon Monoxide Emissions,* Maricopa Association of Governments (*17*);
- *I-5 South HOV Project,* Parsons Brinckerhoff (*18*);
- *Santa Monica Freeway Inbound Research Project,* University of California (*19*);
- *Simulation Analyses of Proposed Improvements for the Southwest Freeway,* Texas Transportation Institute (TTI) (*20*);
- *Simulation Analyses of Proposed Improvements for the Eastex Freeway,* TTI (*21*);
- *Dallas Area High-Occupancy Vehicle Study,* TTI (*22*);
- *I-25 Ramp Metering Final Evaluation Report,* Colorado Division of Highways (*23*);
- *Computer Simulation To Compare Freeway Improvements,* TTI (*24*);
- *Use and Effectiveness of Synthetic Origin-Destination Data in a Macroscopic Freeway Simulation Model,* TTI (*10, 25, 26*);
- FREQ applications and evaluations of QEW Expressway, Ottawa Queensway, and Highway 401, Ontario Ministry of Transportation and Communications (*27-31*);
- Freeway-tunnel approach study in New Jersey, URS Company, Inc.;
- Phoenix I-10/I-17 freeway study, JHK & Associates;
- Evaluation of control and performance on Garden Grove Freeway, University of California; and
- Montreal freeway study, DELCAN Corporation.

THE INTRAS MODEL FAMILY

There has been considerable activity with INTRAS during the 1980s. These activities will be described in three parts: development and testing, applications, and further development and testing.

KLD Associates completed their development and testing of the INTRAS model in 1980 with a series of reports describing this development and validation (*32-34*). The model was applied in an investigation of the effect of location of freeway traffic sensors on incident detection and in the evaluation of control strategies in response to freeway incidents (jointly with Orincon Corporation) (*35-37*).

There have been a number of reported applications of INTRAS by other organizations and one draft report describing an application was located in the literature search (*38*). The reported applications include the following:

- Energy conservation studies, JFT Associates (*38-40*);
- Evaluation of reconstruction of a Detroit freeway, Michigan State University;
- Evaluation of the effect of truck accidents, University of California, Irvine;

- Jones Falls Freeway study in Baltimore, KLD Associates;
- QEW Freeway study in Toronto, Ontario Ministry of Transportation and Communications (*7, 28, 30, 41*);
- Fourteenth Bridge study in Washington, D.C., FHWA;
- Kennedy Expressway Study in Chicago, FHWA;
- Theodore Roosevelt Bridge study in Washington, D.C.; FHWA (*42*);
- Minneapolis Freeway KRONOS simulation program testing, University of Minnesota.

Further development and testing of INTRAS continue. In 1982 Bullen reported the development of FOMIS based on INTRAS (*43*). The intent was to overcome some of the traffic operations difficulties with INTRAS, improve model speed, and provide for use on limited-capacity computers. The model was applied to a weaving section on I-95 in Dade County, Florida.

Another major development is currently under way by JFT and Associates, sponsored by FHWA. This will consist of reprogramming INTRAS according to structured design techniques and enhancing it to make it more user-friendly and applicable to a wider range of applications. The revised model will be called FRESIM and will be incorporated into the TRAF family of programs being developed by FHWA.

THE MACK MODEL FAMILY

Several applications in the early 1980s revealed the need for some improvements of FREFLO (earlier versions were called MACK), particularly in the modeling of congestion when capacities or demand, or both, along a freeway changed significantly. Two earlier applications are presented first and then the work of several research teams attempting to analyze and improve FREFLO will be discussed.

TTI and Daro Associates undertook an NCHRP project to develop guidelines for the selection of ramp-control systems (*44*). Extensive use was made of MACK in assessing the effect of ramp control. About the same time the Ontario Ministry of Transportation and Communications applied FREFLO to the QEW Freeway near Toronto in an attempt to calibrate and validate the model (*45*). The overall conclusion was that the model exhibited instabilities and did not track real-world data correctly.

The author of MACK performed additional work on FREFLO, with particular attention to discontinuity in the equilibrium relationship between speed and density (*39*). The author reported a greatly improved quality of FREFLO predictions using the results of this research.

In 1980 a group of researchers at the University of California, Berkeley (UCB), sponsored by the California Department of Transportation and FHWA, began work on the first of a series of three research projects dealing with dynamic traffic-responsive control strategies for freeways. FREFLO was selected as a starting point, extensively modified, and renamed FRECON (*46-50*). The modifications included automatic selection of subsection lengths (to overcome the earlier-identified problem with modeling congestion), the output of point detector signals, and the incorporation of pretimed and local traffic-responsive entry control algorithms. The model was applied to the Santa Monica Freeway and tested on the Ottawa Queensway by the Ontario Ministry of Transportation and Communications.

The second UCB research effort emphasized segmentwide control and corridor evaluation (*51-54*). The major outcome of this research included the development of a freeway corridor model (named FRECON2), development and evaluation of segmentwide traffic-responsive freeway entry control strategies, and field implementation guidelines. The model was applied to the Santa Monica Freeway, and a user's manual is available (*55*). The Ontario Ministry of Transportation and Communications experienced problems in their application and encountered excessive CPU time.

The current research effort is concerned with developing an on-line algorithm for determining when detector data are acceptable and responsive control strategies under incident conditions (*48, 56*). Complementing this research effort, one researcher has suggested further possible improvements to FRECON2 (*57*) and another researcher has suggested another model formulation (*58, 59*). Current plans call for the application of FRECON2 to the Garden Grove Freeway in Southern California and to the San Francisco-Oakland Bay Bridge.

Under support from FHWA, KLD Associates recently implemented some modifications in the FREFLO formulation designed specifically to resolve its difficulties in properly representing congested conditions (*60*). The simulation results for some test and actual networks indicate that the model is now capable of describing moderate and severe freeway congestion.

Further development of FREFLO is currently under way by JFT and Associates, sponsored by FHWA. No reports were located in the literature search describing this activity.

THE SCOT MODEL FAMILY

In the early 1980s Reiss et al. described the traffic control algorithm development for SCOT (*61*). Also in the early 1980s it is believed that SCOT or a refined version was applied to the Long Island Expressway as part of the IMIS project. However, the literature search did not reveal any published papers.

Review of the literature and discussions with several freeway modelers have not identified any further development in applications with the SCOT model family since the early 1980s.

OTHER RECENTLY DEVELOPED MODELS

Several additional models are currently available that have been developed either as new models or as significantly modified, previously reported models. These additional models are FREESIM, KRONOS, TRAFLO, and ROAD-RUNNER.

FREESIM, developed at Ohio State University, is a microscopic simulation model designed particularly for evaluation of the effects of freeway lane closures. A number of papers describe the development and application of FREESIM (*62-66*). There is no evidence that the model has been applied by others.

After studying several existing models, a research team at the University of Minnesota developed a new one, KRONOS. They wanted to develop a new model that would be efficient in structure and formulation but include the treatment of merging, weaving, and diverging traffic. A significant number of papers have been published and the most recent ones describe an interactive, menu-driven microcomputer version (*67-78*). In addition to the applications by the developers, KRONOS has been applied to a section of the Ottawa Queensway and possibly used by the Minnesota Department of Transportation. Following experimentation and testing, a new version of the KRONOS program (version V) is now being developed and will be released in early 1987. This version includes improved graphics, collector-distributor (C-D) roads, construction zones, left-hand side entrances and exits, and other geometric and demand complexities. The program is also being extended to corridors.

TRAFLO is actually a set of five components that integrates traffic simulation with traffic assignment. The traffic simulation portion includes options for modeling freeways, corridors, urban and suburban arterials, and grid networks. A number of papers are available that describe the development, application, and user's guide (*79-83*). TRAFLO-M is an extended version of TRAFLO that substitutes the DYNEV submodel for the FREFLO submodel and adds the ability to simulate ramp-metering strategies (*84-88*).

The ROADRUNNER freeway model was developed at the University of Toronto for the Ontario Ministry of Transportation and Communications in 1978 (*89*). The model is intended to be used to characterize global system performance and is macroscopic in nature, dealing with average quantities of flow, density, and speed. ROADRUNNER is an attempt to join the use of the numerical integration approaches of MACK with the hydrodynamic theory of FREQ. There is no evidence that the model has been applied by others.

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