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Diversion of Freeway Traffic in Los Angeles: It Worked

D.H. ROPER, R.F. ZIMOWSKI, and A.M. IWAMASA

ABSTRACT

The California Department of Transportation (Caltrans) implemented a traffic management plan aimed at bringing about voluntary traffic diversion upstream of a section of freeway that needed to be closed for 6 hr for maintenance operations. Without an extensive traffic management effort, delays of more than 2 hr were anticipated. The plan was designed to limit delays on the affected high-volume freeways (160,000 to 225,000 average daily traffic) to a maximum of 20 min. The plan included an aggressive public information campaign before the closure by using both media and freeway signing and an extensive use of changeable message signs during the operation. Significant diversion from two freeways that feed the closure area was achieved. Congestion extended about 2.5 miles upstream of the closure at its maximum; actual delays never exceeded the targeted 20 min. The plan, how it was developed, and how it was implemented are described. Results of the operation are also presented.

How can adequate working space and time be provided to safely conduct activities on operating freeways in metropolitan areas without causing extensive traffic tie-ups? It is not a unique problem, but it was one that faced the California Department of Transportation (Caltrans) in May 1982 as plans were being made to replace lighting fixtures located within a tunnel on the Santa Ana Freeway near downtown Los Angeles.

The Santa Ana Freeway is a main artery from the suburbs of Orange County and southeastern Los Angeles County into and through central Los Angeles (Figure 1). Near its junction with the San Bernardino Freeway (where the maintenance work needed to be done), the inbound freeway consists of a two-lane tunnel section with no shoulders. Immediately upstream of the tunnel the Santa Ana Freeway consists of three lanes that carry about 120,000 vehicles per day. Each work day heavy congestion, with average speeds of about 10 mph, are common from about 7:00 to 9:00 a.m. During the midday off-peak hours the freeway is free of congestion, operating at about 60 percent of its capacity and carrying more than 3,000 vehicles per hour.

The required maintenance work would take about 6 hr to complete. It was not possible to keep the freeway partially open and, at the same time, provide the clearances between workmen and traffic prescribed in the safety regulations; the roadway is too narrow to do both things. Previously, maintenance on this section of freeway had been performed during those hours of low traffic volumes--either during the night or on weekends. Neither of these times was considered a satisfactory option for this operation; night work under traffic conditions is potentially hazardous, and it was desired to avoid the higher costs associated with weekend work. In addition, if the freeway could be fully closed, other needed maintenance work on traffic striping, raised pavement markers, signs, and general roadwork would be accomplished. Full closure of the inbound freeway from the East Los Angeles Interchange to the San Bernardino Freeway on a weekday was, from a maintenance operation standpoint, a desirable course of action.

There was a key traffic question: Could the freeway be closed without causing massive traffic jams that could affect a major portion of the downtown freeway system?

For several years Caltrans has had an extensive program to actively manage traffic through and

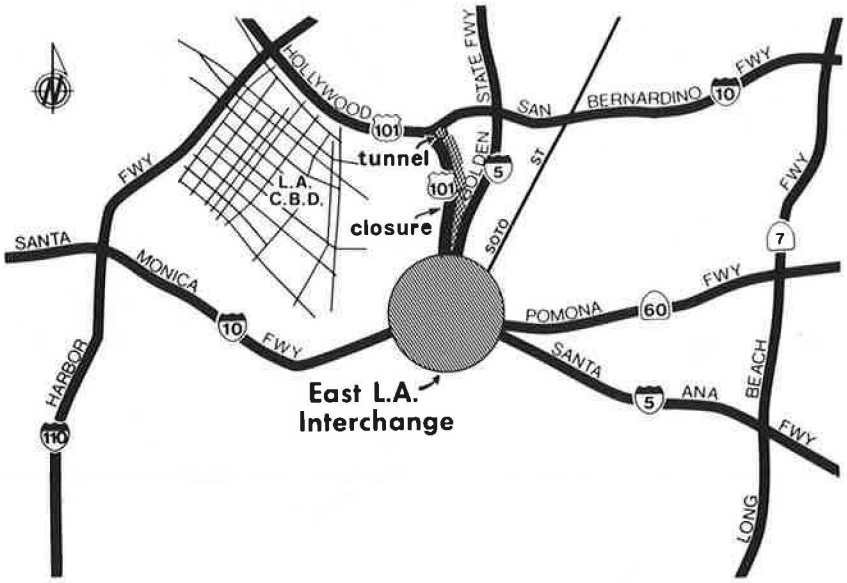

FIGURE 1 System map.

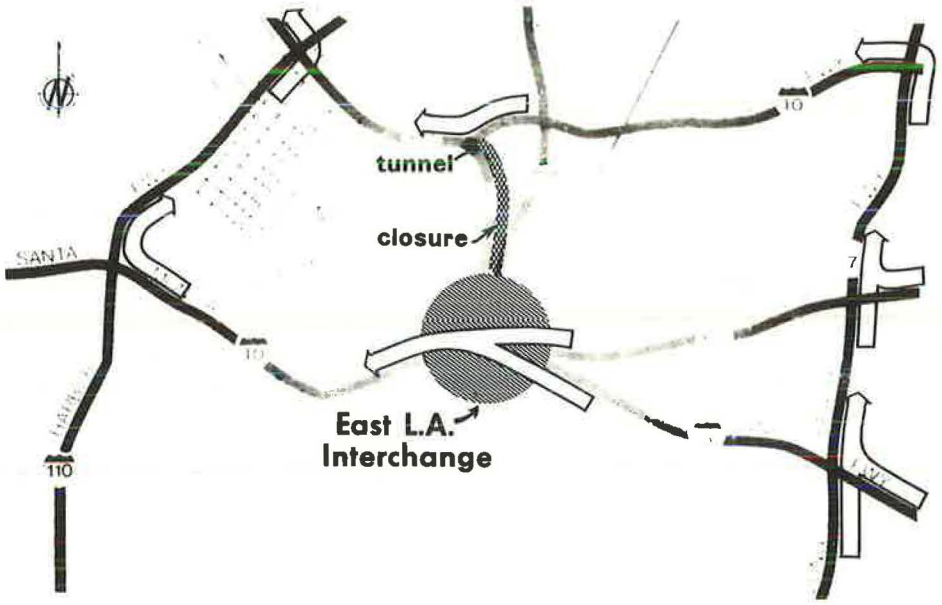

FIGURE 2 Planned diversion routes.

around freeway closures in the Los Angeles region. Based on that experience, and the anticipated reaction of the public to a midday closure in this location, it was generally concluded that a maximum delay of about 20 min would not be unreasonable. An analysis was then undertaken, looking at several alternatives, to develop a plan to manage the anticipated traffic and to keep delays to less than 20 min throughout the day.

A plan to establish a limited closure through the work site and to detour traffic to local surface streets or to other freeways in the immediate vicinity of the closure was not feasible. Alternate capacity to serve the historic demands throughout the day simply did not exist, nor could freeway offramps in the area handle the projected traffic loads. A capacity/demand analysis indicated that traffic on the freeways leading into the closure would be heavily congested for 7 miles, with expected delays to the motoring public of 2 hr or more.

A second plan (the one ultimately adopted) focused on closure of about 2 miles of the freeway between the East Los Angeles Interchange and the San Bernardino Freeway, on reducing the traffic approaching the area through diversion to other freeways upstream of the closure, and on providing alternate freeway routes for that traffic that approached the closure (Figure 2).

The area to be closed is fed by two routes--the Santa Ana Freeway and the Pomona Freeway. Approaching the East Los Angeles Interchange, the Santa Ana Freeway carries about 225,000 vehicles per day; at mid-day, hourly volumes of 6,500 vehicles are common (Figure 3). About 35 percent of this traffic, or 2,300 vehicles per hour, stays on the Santa Ana Freeway through the East Los Angeles Interchange and proceeds toward Los Angeles. The Pomona Freeway, which carries about 160,000 vehicles per day, contributes approximately 1,000 vehicles per hour during mid-day to the closure area. If traffic management was to be successful, and if the 20-min delay criterion was to be met, it was estimated that it would be necessary to reduce the hourly volumes approaching the closure on the Santa Ana Freeway and on the Pomona Freeway to about 5,000 and 2,800, respectively. If this could be accomplished the Santa Monica and Harbor freeways could handle the extra traffic load expected to be placed on them.

There was still a question regarding diversion: Would motorists on several freeways (not just the Santa Ana Freeway) voluntarily divert to other freeways at points several miles upstream of the actual closure? Adding to the uncertainty was the fact that those motorists had to be persuaded to take alternate routes even though no congestion from the closure might exist at their point of diversion. Prior experience in handling traffic at freeway closures resulting from major incidents led to the conclusion that the needed diversions could be achieved through the combined use of

1. A comprehensive public information campaign before the closure, and
2. The aggressive management of traffic, including extensive use of changeable message signs, during the closure.

Several days before the operation press releases were issued and information regarding the closure was included in traffic advisories furnished on a regular basis to radio stations throughout the Los Angeles region. The plan was given satisfactory coverage by both newspapers and radio. In addition, on the day before the closure motorists were advised of the plan by large signs placed along the section of freeway to be closed.

Maintenance, enforcement, and traffic operations

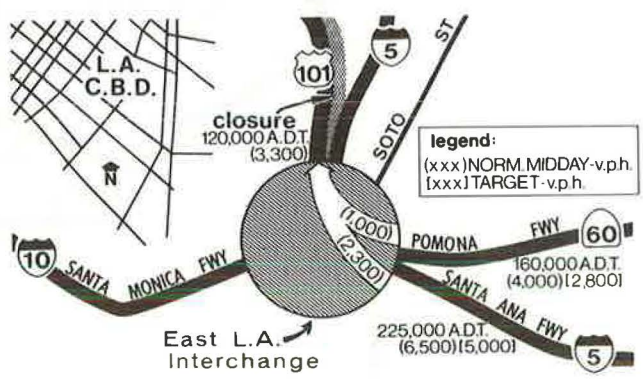

FIGURE 3 Volumes to be diverted.

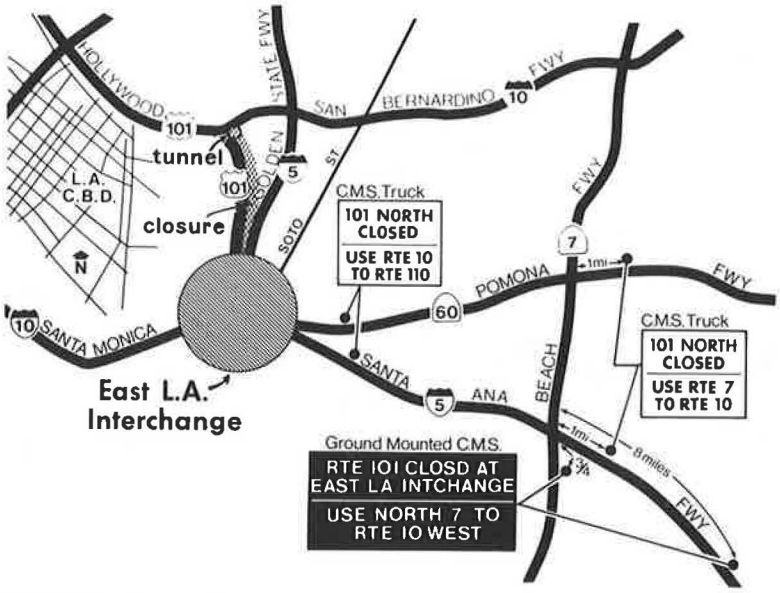

FIGURE 4 Changeable message signs.

personnel who would be involved in the implementation of the traffic management plan held a series of coordination meetings before the closure, and there was a full understanding of the overall strategy. Contingency plans to modify the operation to fit actual conditions as they developed, or to terminate the operation, were made.

Operations to begin closure were undertaken after the peak period and were completed at about 10:00 a.m. Several changeable message signs were activated to encourage the needed diversion (Figure 4). Other truck-mounted changeable message signs were used to warn approaching traffic of the end of queues. Extensive monitoring of the operation was performed by using both ground units and a helicopter. Teletype messages, this time presenting the actual traffic conditions, continued to be provided to radio stations.

The operation was carried out smoothly, with no major problems. Significant diversions to other freeways did take place, although not as great as was estimated. Congestion, however, was held to manageable levels, and delays were about what had been expected (Figure 5).

Traffic approaching the closure on the Santa Ana Freeway was reduced by about 16 percent, to 5,500 vehicles per hour. Two diversions contributed to this: traffic coming in the Santa Ana Freeway turned northward on the Long Beach Freeway (500 vehicles per hour), and many of those on the Long Beach Freeway continued northward instead of turning onto the Santa Ana Freeway (500 vehicles per hour).

It was observed that there was only a slight diversion of traffic from the westbound Pomona Freeway to the northbound Long Beach Freeway. A nominal increase in volumes flowing from the northbound Long Beach Freeway to the westbound Pomona Freeway was also observed. Thus there was virtually no change in the volume on the Pomona Freeway approaching the East Los Angeles Interchange (Figure 5). This condition was attributed to two factors:

1. There was no signing on the northbound Long Beach Freeway approaching the Pomona Freeway to ad-

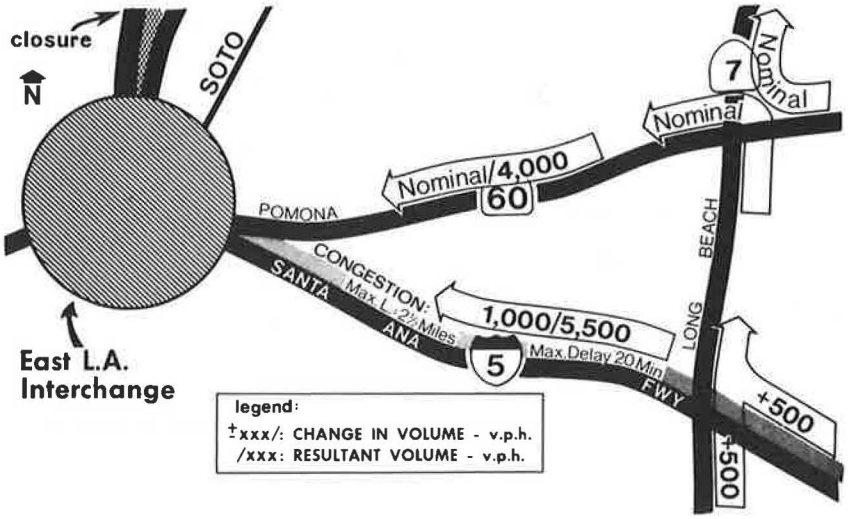

FIGURE 5 Upstream results: diversions.

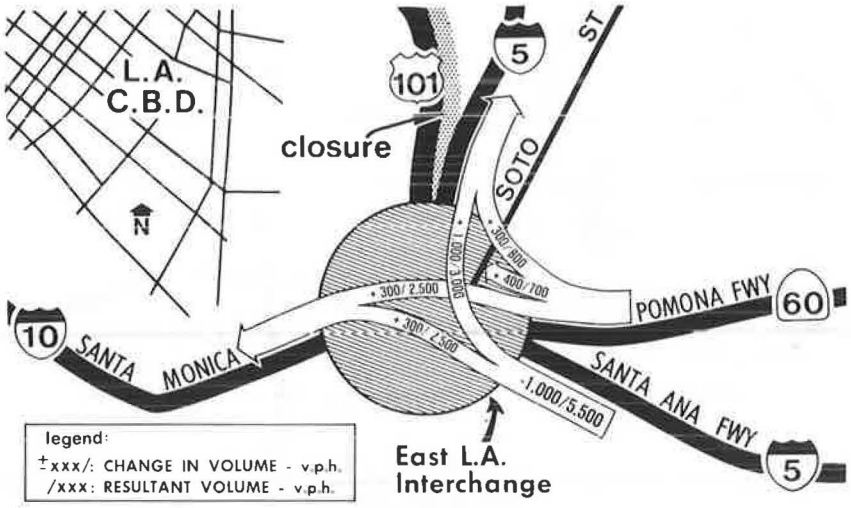

FIGURE 6 Results: diversions at East Los Angeles Interchange.

vise Los Angeles-bound motorists to continue northward to the San Bernardino Freeway, and

2. Congestion, which could have encouraged diversion, never occurred at the Long Beach/Pomona Interchange.

The overall result of the diversion upstream was that the traffic to be detoured around the closure was reduced by more than 30 percent, from 3,300 to 2,300 vehicles per hour.

There was an unanticipated redistribution of traffic from the Santa Ana Freeway at the East Los Angeles Interchange, with more traffic going to the Golden State Freeway (+1,000) than to the Santa Monica Freeway (+300), even though signing directed traffic to the Santa Monica Freeway (Figure 6). This was attributed to the geometrics of the closure, which were such that those who were caught in the congestion chose to escape onto the Golden State Freeway.

There was also some increase in the volumes on several off-ramps in the area (particularly to Soto Street), as motorists attempted to avoid congestion by leaving the freeway system altogether.

Although upstream diversion was not what had been hoped for, the unanticipated flows to the Golden State Freeway and to Soto Street provided the necessary relief that kept traffic moving reasonably well, which avoided termination of the operation. No problems developed on those freeways onto which traffic was diverted.

Congestion and maximum delays were about what had been expected, although the duration of congested conditions was less than anticipated. On the Santa Ana Freeway congestion built immediately after closure and extended back a maximum distance of 2.5 miles. Maximum delays, recorded through the use of tachometer-equipped cars that flowed with traffic, were about 20 min (Figure 5). This condition gradually dissipated throughout the morning and, by noon, a free-flow condition was reached; the freeway remained essentially free of congestion until the operation concluded at 3:30 p.m. On the Pomona Freeway free flow was maintained throughout the operation. Some localized congestion resulted at the Soto Street off-ramp.

The success of this operation can be attributed to several elements that were brought together to achieve the needed levels of diversion: development of a sound plan, mobilization of personnel and equipment to implement and monitor the operation, informing the public before the closure through media coverage and signing, use of changeable message signs during the operation, actively managing traffic during the event, and cooperation of the public. It was critically important that convenient, easily understood, straightforward alternate freeway routes existed.

It is highly doubtful that the operation could have been successfully conducted if diversion to local surface streets had been called for. Caltrans' experience in Los Angeles has established that there is a distinct reluctance on the part of the motoring public to voluntarily divert from the freeway system to surface streets, regardless of how adequate those surface streets might be.

As freeway systems age, the need for extensive maintenance and rehabilitation work, and the resulting impact on traffic, is increasing dramatically. At the same time traffic demands are burgeoning and there is a relatively low tolerance on the part of the public to disruption of their normal traffic patterns and levels of service.

This experience has clearly demonstrated that, given certain conditions, these seemingly competing needs can both be successfully met through the use of proven traffic management techniques.

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Developing Segmentwide Traffic-Responsive Freeway Entry Control

SUK JUNE KAHNG, CHAWN-YAW JENG, JAMES F. CAMPBELL, and ADOLF D. MAY

ABSTRACT

As part of a Highway Planning and Research project two types of segmentwide traffic-responsive freeway entry control strategies were developed: extended local traffic-responsive control and extended pretimed traffic-responsive control. These strategies were evaluated on a macroscopic dynamic freeway corridor simulation model by using 3 days of traffic data from the Santa Monica Freeway in Los Angeles. Extended local traffic-responsive control had a consistent advantage over extended pretimed traffic-responsive control. Based on the evaluation the extended local traffic-responsive control strategy has been selected for further evaluation and future field implementation on a segment of the Santa Monica Freeway. The development and evaluation of the segmentwide traffic-responsive control strategies are described. A brief description of implementation plans is also included.

Since its appearance in the early 1960s (1-6), freeway entry control, or ramp metering, has become a vital tool for transportation engineers to improve freeway operation. During the past two decades a number of different freeway entry control strategies have been developed and implemented (7-9). They may be classified into the following three categories: pretimed (10-15), local traffic-responsive control (16-21), and segmentwide traffic-responsive control (22-25).

Although much research and implementation has taken place in the first two categories, not as much experience has been acquired in the third. Recognition of this fact, coupled with anticipated advancements in computers and communication technology, identifies research needs with segmentwide traffic-responsive control.

Based on these observations, a freeway research study is being conducted at the Institute of Transportation Studies (ITS), University of California, Berkeley. Major objectives of the research include development of segmentwide traffic-responsive control strategies, field implementation and evaluation of the most promising strategy, and preparation of preliminary guidelines for segmentwide traffic-responsive control.

The purpose of this paper is to discuss current findings and results of the research (26), which are (a) development of the extended freeway corridor model FRECON2, (b) development of two types of segmentwide traffic-responsive control strategies, and (c) evaluation of these control strategies through simulation by using 3 days of traffic data from the Santa Monica Freeway in Los Angeles. A brief discussion of the field implementation plan is also included.

FREEWAY CORRIDOR SIMULATION MODEL: FRECON2

The iterative nature of control strategy development (i.e., strategy formulation, testing, evaluation, modification) requires a reliable freeway corridor simulation model. Desirable attributes of the model for this study include simulating the freeway dynamically, generating point-detector surveillance data, modeling priority entry control, modeling alternative surface streets, and modeling driver's spatial diversion phenomena.

Because no single existing freeway simulation model contains all the attributes required by this study, an existing simulation model was extended for the purpose of the study. The selected model (FRECON) is a macroscopic dynamic freeway simulation model developed by Babcock during a previous study at ITS (9). FRECON evolved from FREFLO, which was developed by Payne (27). Major features of FRECON that distinguish it from FREFLO include

1. An adaptive module that internally determines proper spatial and temporal step sizes to solve the model's discrete freeway state equations; this eliminates the deficiencies of FREFLO when it was applied to a lane-drop bottleneck (28); and
2. The ability to generate surveillance traffic data from emulated point detectors rather than using subsection average traffic performance for surveillance data as in FREFLO.

For the purpose of this study, the FRECON model was further extended into a freeway corridor model, FRECON2 (26). Three major areas of extension of the model include the modeling of priority entry control, alternative surface streets, and driver's spatial diversion. These additional features are briefly described in the following subsections.

Priority Entry Control

There exist three types of freeway entry control distinguished by the entry preference, based on passenger occupancy, given to vehicles at on-ramps (29): normal entry control (NEC), priority entry control (PEC), and no control (NC).

In NEC all the vehicles wishing to enter the freeway are metered by the signal at the on-ramp. In PEC a preset passenger occupancy cut-off value (PCV) is used to divide vehicles coming to the on-ramp into two groups: high-occupancy vehicles (HOVs) and non-HOVs. HOVs are permitted to enter the freeway without delay at the signal, whereas non-HOVs must wait at the signal. In NC all the vehicles are free to enter the freeway without delay, regardless of their passenger occupancies.

Based on these observations, FRECON2 uses a generalized priority entry control concept. NEC and NC are regarded as special cases of PEC. Suppose m is the highest passenger occupancy found among vehicles approaching an on-ramp. Then NEC is equivalent to PEC, with PCV = m + 1. NC is the same as PEC with PCV = 1, where all the vehicles are regarded as HOVs. With this generalized concept of PEC, FRECON2 can model a study section with mixed mode entry control (i.e., a study section with NEC, PEC, or NC).

Driver's Spatial Diversion

Inducing the proper amount of diversion without adversely affecting alternative routes is important for successful ramp metering. By using the diversion formula given in Equation 1, FRECON2 predicts the magnitude of diversion caused by freeway entry control. In the formula the two major factors that influence driver's diversion were chosen to be the percentage travel time difference between the freeway and alternative route (RATIO) and the driver's sensitivity (S):

$$FDIV = (1/2)\left(1 + \sin\left\{\pi[RATIO^S - (1/2)]\right\}\right) \quad (1)$$

where

FDIV = fraction of queued vehicles at an on-ramp that divert to the available alternative routes (i.e., percentage diversion);
RATIO = ratio of ΔT to FT + RD (i.e., percentage travel time difference);
S = driver's sensitivity to the travel time difference between freeway and alternative routes;
ΔT = FT + RD − AT; if ΔT < 0, ΔT is set to 0;
FT = freeway travel time;
RD = delay at on-ramp; and
AT = alternative route travel time.

Based on the diversion formula given in Equation 1, Figure 1 shows a sample relationship between percentage travel time difference (RATIO) and percentage diversion (FDIV) for varying values of driver's sensitivity (S).

Alternative Surface Streets

The major purpose of including an arterial model in FRECON2 is to evaluate (or predict) the potential impact of diverted vehicles on the alternative surface streets. This requires a surface street model that allows travel time to increase as traffic volume increases. The selected model is the Davidson model, which has been used in the FREQ series (29). In the Davidson model the travel time along a section of an arterial is estimated as a function of free flow travel time and flow/capacity ratios, as shown in Equation 2:

$$t = t_0 \cdot \left\{1 + J \cdot [(q/c)/(1 - q/c)\right\} \quad (2)$$

where

t = section travel time,
t_0 = section travel time at free flow speed,
J = Davidson parameter,
q = traffic flow (vehicles per hour), and
c = arterial (or road) capacity (vehicles per hour).

Because of the inherent differences of the freeway model in FRECON and the Davidson arterial model (i.e., the former is a dynamic model whereas the latter is a static one), some consideration should be given to the use of different evaluation intervals for freeway and alternative routes. In FRECON2 the freeway evaluation interval is user supplied with almost no restriction. However, the arterial evaluation interval is internally determined to be the shortest possible interval that still gives enough time for the diverted vehicles to travel to their destination.

SEGMENTWIDE TRAFFIC-RESPONSIVE CONTROL STRATEGIES

To develop implementable (or feasible) segmentwide control strategies, in terms of available hardware and computing capability, the current study was directed toward the development of the following two types of control strategies (26): extended local traffic-responsive (ELT) control and extended pretimed traffic-responsive (EPT) control. These two strategies were then tested on the FRECON2 model by using 3 days of traffic data from the Santa Monica Freeway in Los Angeles. An overview of the strategies is given first, followed by the results of evaluation through simulation.

ELT Control Strategy

The ELT freeway entry control strategy has evolved from local traffic-responsive (LT) freeway entry control. The major difference between them is that the ELT control has an extended view of the freeway, so that each on-ramp controller is aware of, and reacts to, the changing traffic situation at its neighboring on-ramps.

Figure 2 shows an overview of the ELT control scheme. In the ELT control on-line traffic information is collected from detectors on both the freeway main line and the on-ramps and off-ramps. Micropro-

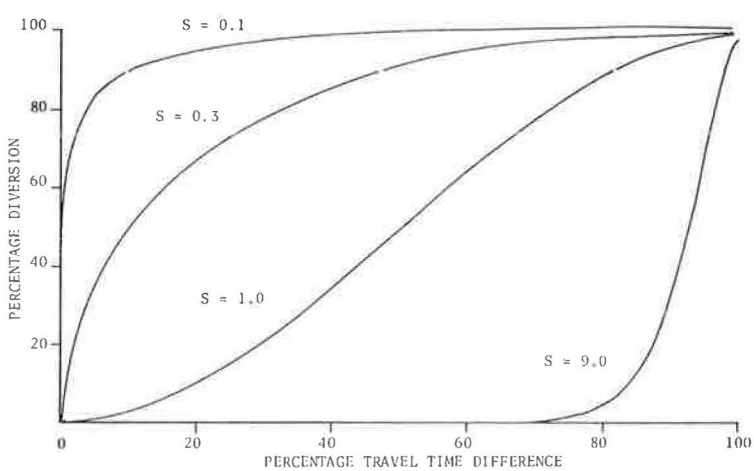


FIGURE 1 Relationship between percentage travel time difference (RATIO) and percentage diversion (FDIV) for different driver sensitivity values (S).

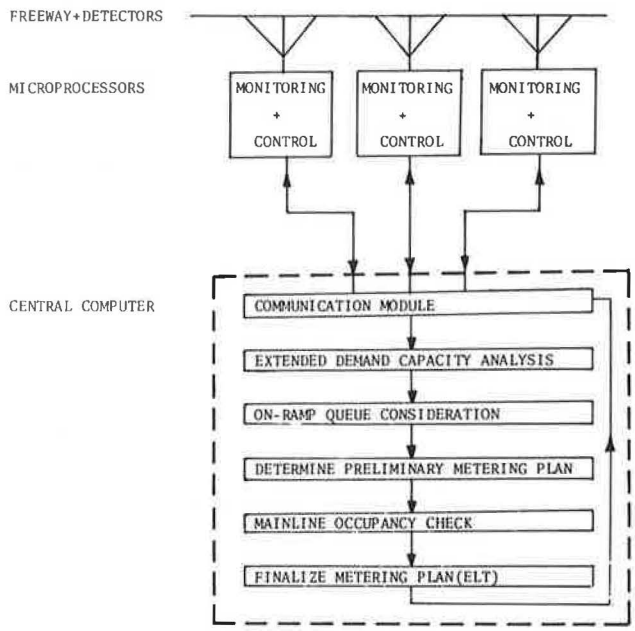


FIGURE 2 Overview of ELT control strategy.

cessor-based controllers (e.g., 170-type controller) located near the on-ramps transmit this information to the central computer.

Based on the real-time traffic data, ELT control determines the metering rates for each on-ramp to operate freeway bottlenecks at capacity while preventing main-line congestion and giving due consideration to on-ramp queues. To accomplish this purpose the ELT control uses the following five major steps: extended demand capacity analysis, on-ramp queue consideration, determination of preliminary metering plan, main-line occupancy check, and finalize metering plan.

Step 1: Extended Demand Capacity Analysis

For each control period t, once the central computer receives on-line traffic information from the microprocessor, extended demand capacity analysis estimates the effective downstream main-line capacity of each on-ramp i. The effective downstream main-line capacity of on-ramp i at time t, EDC(i,t), is defined as the minimum capacity available at time t between on-ramp i and the next downstream on-ramp i + 1 (including the section containing the on-ramp i + 1). This considers not only off-ramp flows between the on-ramps but also the minimum metering rate constraint of on-ramp i + 1.

The outcome of the first step is EDC(1,t), ..., EDC(n,t), where n is the number of on-ramps in the freeway segment.

Step 2: On-Ramp Queue Consideration

To balance out the queue length at each on-ramp, on-ramp i is allowed to ask for help from the upstream on-ramp i - 1 (i.e., on-ramp i can request on-ramp i - 1 to reduce its ramp metering rate). However, in the field the exact queue length at the on-ramp is not usually available to the controller once the queue grows beyond the queue detector. Thus the amount of help asked by on-ramp i from upstream on-ramp i - 1 is based on the following formula:

$$HELPQ(i,t) = A \cdot [ONVMAX(i) - ONVOLR(i,t)] + B \quad (3)$$

where

A = user-supplied design parameter (currently, A = 0.9 is used),
B = user-supplied design parameter (currently, B = 300 is used for a single lane on-ramp),
ONVMAX(i) = maximum metering rate at on-ramp i,
ONVOLR(i,t) = metering rate at the on-ramp i during control period t, and
HELPQ(i,t) = reduction in the metering rate of upstream on-ramp i - 1 requested by on-ramp i at control period t.

The outcome of the second step is HELPQ(1,t), ..., HELPQ(n,t).

Step 3: Determination of Preliminary Metering Plan

Effective downstream main-line capacities of each on-ramp i, EDC(i,t), calculated in the first step are reduced by the amount of HELPQ(i + 1,t) requested by downstream on-ramp i + 1. Then the preliminary metering rate of on-ramp i for the next control period t + 1 is set as the difference between the reduced effective downstream capacity and the on-line measured main-line traffic flow upstream of on-ramp i:

$$PMR(i,t+1) = [EDC(i,t) - HELPQ(i+1,t)] - MF(i,t) \quad (4)$$

where PMR(i,t + 1) is the preliminary metering rate for on-ramp i during the next control period t + 1, and MF(i,t) is the on-line measured main-line flow immediately upstream of on-ramp i during control period t.

The outcome of the third step is PMR(1,t + 1), ..., PMR(n,t + 1).

Step 4: Main-Line Occupancy Check

The main-line occupancy check is used as a feedback mechanism in the ELT control. This operates on two

levels. The first level compares each on-ramp's upstream main-line occupancy against the preset critical occupancy value. If this comparison indicates that the detector occupancy upstream of on-ramp i is greater than the critical occupancy associated with the location, the PMR(i,t) is reduced to the minimum metering rate for on-ramp i. Otherwise the PMR(i,t) remains unchanged. The second level of occupancy checking allows on-ramp i to request help from upstream on-ramp i - 1, in terms of a reduction in the metering rate of on-ramp i - 1, if the main-line occupancy problems at on-ramp i persist for more than a user-specified number of control periods. Then the PMR(i - 1, t + 1) is reduced to the minimum metering rate for on-ramp i - 1.

The outcome of the fourth step is revised PMR(1,t + 1), ..., PMR(n,t + 1), if necessary.

Step 5: Finalize Metering Plan

The revised preliminary metering rates that result from the foregoing analyses are checked against minimum and maximum metering rate constraints to be finalized. Then the finalized metering rates [MR(i,t + 1)] are sent to the microprocessors at the on-ramps to be implemented.

The outcome of the fifth step is finalized metering rates, MR(1,t + 1), ..., MR(n,t + 1).

This procedure is repeated in the ELT controller for each control period. Currently, a 1-min control period is used in the ELT control.

EPT Control Strategy

The EPT control strategy has evolved from linear programming (LP) based pretimed control strategies (29-31). The major difference is that the EPT control determines the metering rate based on on-line traffic information and historical traffic data rather than based solely on historical data, so the EPT control can respond to changing traffic situations.

Figure 3 shows an overview of the EPT control scheme. In EPT control the only real-time traffic data are traffic flow data from the first main-line section of the freeway. Other traffic flow data needed for the EPT control, which are on-ramp and off-ramp flows, will be historical. Based on these real-time and historical traffic data, EPT control determines metering rates to maximize the sum of input flows while preventing main-line congestion. In order to do this EPT control uses the following three major steps: combining on-line and historical data, demand prediction, and LP optimization.

Step 1: Combining On-Line and Historical Data

For each control period, the central computer receives on-line measured traffic flow of the first main-line section and combines this information with the historical traffic demand stored in the central computer memory for on-ramps and off-ramps, O(i,t + 1) and D(i,t + 1). This step generates a preliminary traffic demand set for the first main line and all on-ramps and off-ramps in the freeway segment.

The outcome of the first step is O(1,t), O(2,t + 1), ..., O(n,t + 1), ..., D(1,t + 1), ..., D(n,t + 1).

Step 2: Demand Prediction

Because of the static nature of the LP technique used in the EPT control, the length of each control interval should be comparable to the time required for vehicles to travel through the control section. In the case of a long control interval, actual traffic flow from the first main line in the next control period, O(1,t + 1), might be substantially different from that of the current control period, O(1,t). Then it is desirable to predict O(1,t + 1) based on O(1,t) and use the best estimate of O(1,t + 1), O'(1,t + 1). For this purpose, several prediction algorithms were tested, including a simplistic approach, a historical factor approach, a moving average approach, and an autoregressive approach (26).

The historical factor approach proved superior. The EPT control uses this historical factor approach (as shown in Equation 5) to predict the first sec-

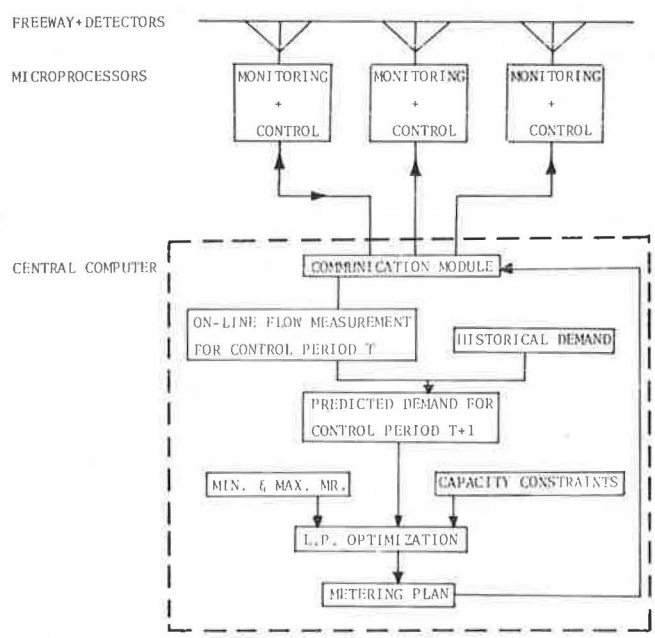


FIGURE 3 Overview of EPT control strategy.

tion's main-line traffic demand for the next control period:

$$O'(1,t+1) = O(1,t) \cdot K(t+1) \tag{5}$$

where

O'(1,t + 1) = predicted traffic flow on the first main-line section for control period t + 1,
O(1,t) = on-line measured traffic flow on the first main-line section during control period t, and
K(t + 1) = historical factor for control period t + 1.

The data in Table 1 give the test results of the historical factor approach that used 3 days of traffic data from the Santa Monica Freeway.

The outcome of the second step is O'(1,t + 1).

Step 3: LP Optimization

By using the predicted first section's main-line traffic demand and the historical data for on-ramps and off-ramps, LP is used to generate optimal metering rates that maximize the sum of input flows within constraints of section capacities. Equations 6-10 give the formulation used in the EPT control. Maximum and minimum metering rate constraints are also considered in this stage.

$$\text{Maximize} \sum_{i=1}^{n} x_i \tag{6}$$

TABLE 1 Test Results of Historical Factor Approach

	Mean	Standard Deviation	Correlation Coefficient	t Value	F Value
Day 1			0.801	-0.11	1.100
R	7,150	551			
H	7,157	579			
Day 2			0.842	-0.08	1.095
R	7,033	713			
H	7,038	745			
Day 4			0.741	0.22	1.695
R	6,631	481			
H	6,617	625			

Note: R = real data [O(1,t)] in vehicles per hour, and H = data predicted by the historical factor approach [O'(1,t)] in vehicles per hour.

$$\text{Subject to} \sum_{i=1}^{n} A_{ij}x_i \leq C_j \quad \text{for } j = 1, 2, ..., m \tag{7}$$

$$x_i \leq \text{ONVMAX}(i) \quad \text{for } i = 1, 2, ..., n \tag{8}$$

$$x_i \geq \text{ONVMIN}(i) \quad \text{for } i = 1, 2, ..., n \tag{9}$$

$$x_i \geq 0 \quad \text{for } i = 1, 2, ..., n \tag{10}$$

where

x_i = input flow rate from on-ramp i (i.e., the finalized metering rate for on-ramp i),
n = number of on-ramps,
m = number of freeway sections,
A_{ij} = fraction of vehicles from on-ramp i that travel through section j,
C_j = capacity of section j,
ONVMAX(i) = maximum metering rate for on-ramp i, and
ONVMIN(i) = minimum metering rate for on-ramp i.

The outcome of the third step is finalized metering rates MR(1,t + 1), ..., MR(n,t + 1).

The finalized metering rates are sent to microprocessors in the field to be implemented. This procedure (steps 1-3) is repeated in the EPT control for each control period. Currently, a 5- to 10-min control period is used in the EPT control.

Preliminary Evaluation of Extended Control Strategies

The ELT and EPT control strategies were tested with the FRECON2 model. Because the major purpose of the testing was to select one strategy for more comprehensive evaluation and field implementation, the scope of the evaluation was limited to the expected performance of the strategies on the first day of implementation without diversion. Thus new features of the FRECON2 model (i.e., priority treatment, diversion, surface street modeling) were not engaged in the testing.

Figure 4 shows a schematic diagram of the modeled study section, which is approximately 7.8 miles long. The simulation time period is from 15:00 to 18:00. From the 5 days of traffic data for the Santa Monica Freeway obtained from the California Department of Transportation (Caltrans) during a previous study at ITS (9), 3 days of data (days 1,

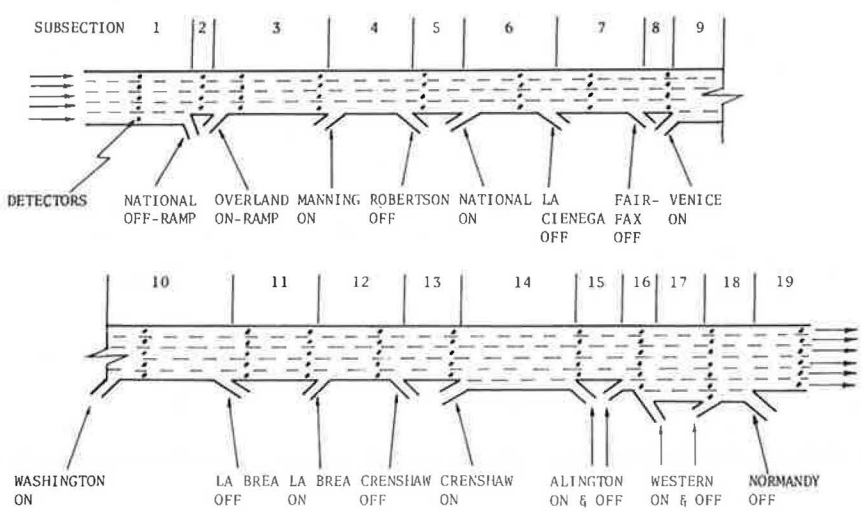


FIGURE 4 Schematic diagram of modeled study section (Santa Monica Freeway, eastbound).

2, and 4) with differing degrees of traffic demand (i.e., heavy, medium, and light) were selected for this test. The control parameters in the control strategies were calibrated for day 1. Then the control strategies were applied to days 2 and 4.

There exist a number of criteria that can be used in evaluating freeway entry control strategies. These include traffic performance measures, congestion elimination, bottleneck utilization, on-ramp queues, reliability (or consistency), data requirements, on-line computing time and memory requirements, and theoretical soundness.

Comparison of Traffic Performance Measures

The data in Table 2 compare traffic performance measures that result from the ELT and EPT controls. Both controls produced almost identical total travel service (i.e., total traveled distance in terms of vehicle miles) for all 3 days.

TABLE 2 Comparison of Traffic Performance Measures

Day	Measure	ELT Control	EPT Control	Difference[a] (%)
1	TTD	173,917	173,844	0.0
	FTT	3,537	3,554	-0.5
	OWT	777	849	-8.5
	TTT	4,314	4,403	-2.0
2	TTD	174,808	174,803	0.0
	FTT	3,624	3,771	-3.9
	OWT	1,031	904	14.0
	TTT	4,655	4,675	-0.4
4	TTD	163,168	163,164	0.0
	FTT	3,232	3,219	0.4
	OWT	92	372	-75.0
	TTT	3,323	3,591	-7.5

Note: TTD = total traveled distance (vehicle miles), FTT = freeway travel time (vehicle hours), OWT = on-ramp wait time (vehicle hours), and TTT = total travel time (vehicle hours).

[a]Difference = (ELT − EPT) · 100/EPT.

However, total travel times, which indicate the effectiveness of the control, were different for the two controllers. For all 3 days the ELT control resulted in shorter total travel times (approximately 0 to 7 percent less) compared with those from the EPT control.

For days 1 and 4 the EPT control caused unnecessary excess delay at on-ramps, which resulted in the longer total travel time compared with that from the ELT control. Although the EPT control gave a shorter on-ramp wait time on day 2, this was offset by the substantial increase in the freeway travel time (FTT).

Comparison of Freeway Congestion Elimination

Figure 5 shows traffic density maps (i.e., number of vehicles per lane per mile) for the two control strategies as an indication of the degree of main-line congestion remaining on the freeway. Among the 19 sections in the study site, the density map of only the first 13 sections (1-13) for time period 15:00 to 17:40 is shown in Figure 5 because no congestion occurred downstream of section 13 or after 17:40. Identified bottlenecks in the study site are sections 4, 6, and 11. In the density map, sections with densities greater than 60 vehicles per lane per section are defined as congested sections. The only exception is the bottleneck section 11, which tends to show densities slightly higher than 60 when operating at capacity.

As can be seen in Figure 5, the ELT control was able to eliminate the main-line congestion upstream of all three bottlenecks for all 3 days. However, the EPT control failed to eliminate the congestion for both day 1 and day 2. Although the EPT prevented congestion on day 4, the excessive on-ramp delay, as already given in the data in Table 2, indicates that the EPT overcontrolled the freeway, thus resulting in longer total travel time.

The data in the following table compare the remaining congestion for both controllers in terms of (minute•miles) of congested region remaining:

	Congestion Remaining (minute•miles)	
Day	ELT Control	EPT Control
1	0.0	7.1
2	0.0	40.4
4	0.0	0.0

Again, the EPT control left congestion on the freeway on both day 1 and day 2, whereas the ELT control eliminated the congested region completely for all 3 days.

Comparison of Freeway Bottleneck Utilization

Traffic volume through a bottleneck section downstream of queued on-ramps indicates whether the controller uses the bottleneck effectively. The data in Table 3 give the duration of traffic flow through the bottleneck at a level greater than or equal to 99 percent of the capacity of the bottleneck (i.e., operating near or at capacity).

Except at bottleneck section 4 in day 2, the ELT control kept all three bottlenecks operating near capacity at least as long as the EPT control.

Comparison of On-Ramp Queue Conditions

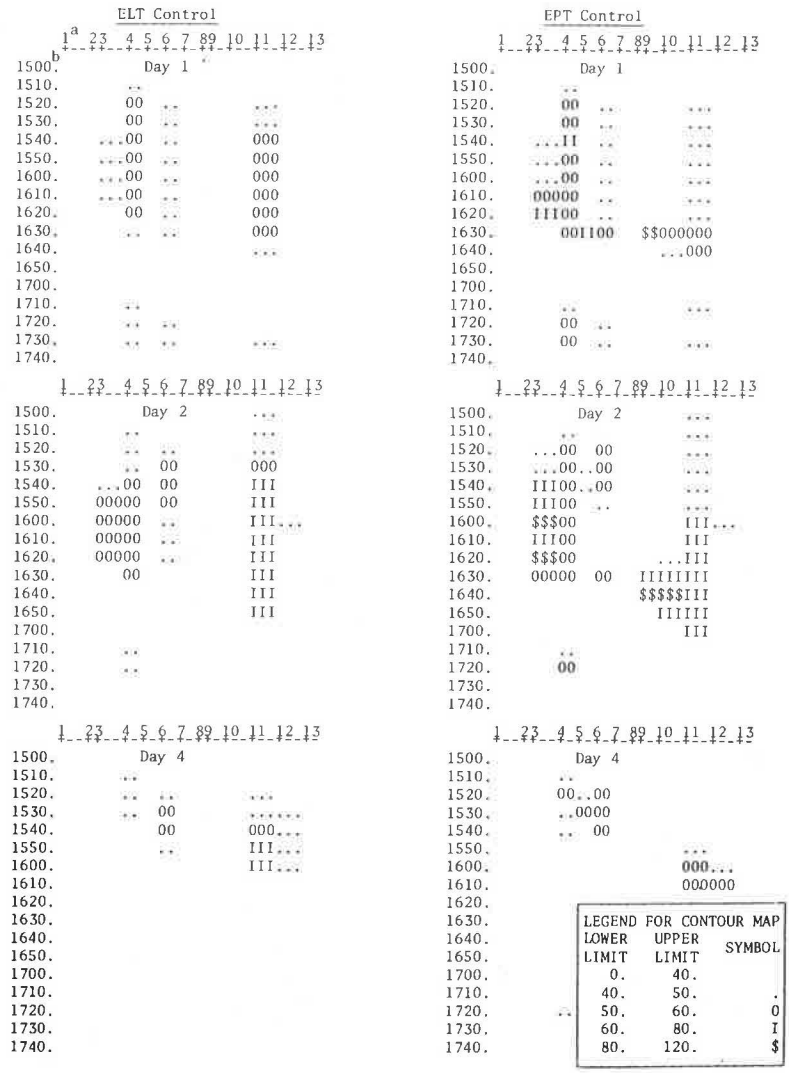
One of the major concerns of practicing traffic engineers when they plan to implement a new control strategy is the on-ramp queue lengths and the consequential impact on the surrounding surface streets. In the simulation tests the ELT control was able to balance out on-ramp queues while eliminating congestion on the freeway. In the EPT control, however, considerable difficulties were experienced in controlling on-ramp queue lengths because of the long control interval required by the static nature of LP in the EPT and the absence of on-line traffic data for on-ramp and off-ramp flows.

The ELT control, compared with the EPT control, requires more observations of traffic on the freeway. Although this might be an additional burden in terms of hardware, this analysis indicated that this frequent on-line observation is necessary for the control to operate effectively and reliably.

In terms of computing time and memory requirements, the EPT control requires more computing time for solving LP and more memory space for storing historical traffic demand.

In addition to the static nature of LP, the absence of both a feedback mechanism and real-time traffic information in the EPT control was identified as a major problem in terms of reliability and effectiveness, especially for different days with varying traffic demand. On the other hand, the ELT control, which has a feedback mechanism and uses a short control interval, resulted in more reliable performance for all 3 days compared with the EPT control.

Throughout this analysis the ELT control had a consistent advantage over the EPT control in the areas of traffic performance, congestion elimina-



[a] Freeway subsection number.
[b] Simulation time period.

FIGURE 5 Comparison of freeway congestion elimination.

TABLE 3 Duration of Near-Capacity Flow Operation at Bottlenecks

		Duration (min)		
Day	Section	ELT Control	EPT Control	Difference[a] (%)
1	4	90	85	5.8
	6	80	75	6.7
	11	80	80	0.0
2	4	70	75	−6.7
	6	55	35	57.1
	11	120	105	14.3
4	4	25	20	25.0
	6	30	20	50.0
	11	40	5	700.0

[a] Difference = (ELT − EPT) · 100/EPT.

tion, bottleneck utilization, on-ramp queue management, and computing time and memory requirement. Based on this analysis the ELT control strategy was selected for further evaluation and future field implementation in a segment of the Santa Monica Freeway. Although the ELT control balanced on-ramp queue lengths for 3 different days, the overall increased queue lengths at on-ramps cause major concerns from the viewpoint of field implementation.

Because the evaluation test was limited to expected queue length on the first day of implementation with no diversion, it is premature to draw any conclusions from those queue lengths. An extensive simulation evaluation of the selected control strategy (ELT) for a wide variety of operating environments is planned before actual field implementation. Major factors to be considered include daily traffic demand variation, on-ramp queue constraints, driver's diversion, alternate routes condition, and so forth. New features of the FRECON2 model will play an important role in this evaluation. Depending on the results of this evaluation, some modifications in the ELT control strategy might be necessary, especially in handling on-ramp queues.

FIELD IMPLEMENTATION PLAN

Field implementation of the ELT control strategy is one important and difficult phase of the project. Based on a field study conducted from May 17 through May 19, 1983, the boundaries of the study section in space and time have been modified. The modified study site boundary chosen for future field imple-

mentation is a 6.4-mile section of the eastbound Santa Monica Freeway in Los Angeles that contains eight on-ramps and eight off-ramps. Because there is no congestion downstream of section 13, the study site was shortened to 6.4 miles (from 7.8 miles). However, because of growth in the traffic demand, the time period has been extended to cover 13:30 to 19:00 (from 15:00 to 18:00). Implementation is expected to be conducted as required hardware and communication systems become available in the study site.

The basic hardware necessary is main-line and ramp detectors, a California Type 170 Controller (32) at each on-ramp, a central computer, and communication lines between each 170 Controller and the central computer. The major problem from the implementation perspective is developing software for the 170 Controller and the central computer based on the ELT control strategy. The study site is currently controlled by a combination of local main-line-responsive (LMR) and time of day (TOD) metering that uses software resident in the 170's developed by Caltrans (33). This software also transmits volumes, occupancies, and error alarms from up to six detectors to the central computer. The central computer is currently used mainly to acquire and manage data from the 170's (34). New software is being developed in assembly language for the microprocessor-based 170's and in FORTRAN for the central computer.

New software for the 170 Controller includes routines to transmit a queue length alarm to the central computer and to check the metering rate received from the central computer. If a metering rate is received by the 170 (i.e., sent from the central computer), that value will be used instead of the value calculated in the existing LMR metering algorithm. If a metering rate is not received because of an error of the central computer or a communication failure, then the LMR metering rate is used.

New software for the central computer includes routines to receive, identify, error check, and store data transmitted from the 170's and to execute the ELT control algorithm to calculate a metering rate. Major tasks in writing the central computer software are the routines to interpret and error check the data received from the 170's. For any responsive control strategy, it is crucial to identify and compensate for errors in the detector data. Extensive testing of the new software must be conducted to ensure the proper and safe operation of the ELT control before actual implementation.

To evaluate the benefits of ELT control a comprehensive traffic study will be conducted before and after implementation. Data to be collected includes main-line and ramp demands, on-ramp delay, on-ramp queue lengths, and traffic volume and travel time along the freeway and alternative surface streets. Two weeks of data collection immediately before implementation form the before study. The after study period consists of the first week of operation of ELT control and another 5 days from the third and fifth weeks of operation.

SUMMARY

Two types of implementable (in terms of available hardware and computing capability) segmentwide control strategies were developed: ELT control and EPT control. To evaluate the extended control strategies an existing macroscopic dynamic freeway simulation model (FRECON) was extended to a freeway corridor model, FRECON2. New features of FRECON2 include the modeling of alternative surface streets, priority entry control, and driver's spatial diversion.

Preliminary evaluation of the ELT and EPT control strategies was conducted on the FRECON2 model by using 3 days of traffic data from the Santa Monica Freeway. Throughout the evaluation the ELT control demonstrated a consistent advantage over the EPT control in decreasing total travel time, eliminating freeway congestion, using the bottlenecks efficiently, and balancing on-ramp queues. Based on the evaluation, the ELT control has been selected for further evaluation through simulation and future field implementation on the Santa Monica Freeway.

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Joint Operational Management of an HOV Facility: A Success in Houston

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ABSTRACT

The Harris County Metropolitan Transit Authority and the Texas State Department of Highways and Public Transportation have jointly managed a combination contraflow and concurrent-flow high-occupancy vehicle (HOV) facility for the past 4 years. The management structure by which the two agencies oversee the daily operations of the facility is discussed. An Operational Agreement and an Operations Plan detail each agency's responsibilities. Of primary importance is the establishment of a Contraflow Project Management Team as the responsible body for daily management of the facility. This successful joint management of the contraflow and concurrent-flow lane has had a positive impact on other interagency transportation-related efforts. It is expected that by 1990 more than 40 miles of HOV treatment will have been jointly constructed by the Metropolitan Transit Authority and the Texas State Department of Highways and Public Transportation.

The Metropolitan Transit Authority of Harris County (METRO) and the Texas State Department of Highways and Public Transportation (TSDHPT) have jointly constructed and operated a combination contraflow and concurrent-flow high-occupancy vehicle (HOV) lane facility to serve the commuting needs of northern Harris County residents. The administrative structure and process by which these separate government agencies jointly manage daily operations of this facility are presented.

PROJECT DESCRIPTION

On August 28, 1979, METRO, in cooperation with TSDHPT, began operation on a contraflow lane on the North Freeway as one element of a comprehensive corridor transportation improvement program. UMTA funded the project with a $2.1 million Sections 5 and 6 Service and Methods Demonstration grant (as provided by the Urban Mass Transportation Act of 1964, as amended).

The contraflow lane operation of the North Freeway extends north from downtown Houston to the North Sheperd interchange, a distance of 9.6 miles. The contraflow lane is available for use by authorized buses and vanpools traveling inbound on the North Freeway between the hours of 6:00 and 8:30 a.m. and outbound between 4:00 and 6:30 p.m.

The North Freeway contraflow lane has a number of unique features that make it a more ambitious project than other HOV contraflow lane projects. Specifically, these include the following:

1. At 9.6 miles, the project is the longest attempted;
2. The project operates during both morning and afternoon peak periods; and
3. The project is available only to vehicles that are authorized in advance.

On March 30, 1981, the existing contraflow lane operation was enhanced by the addition of a 3.3-mile concurrent-flow lane (morning inbound only). The concurrent-flow lane terminates at the contraflow lane entrance at North Shepherd and extends morning priority treatment north to West Road, approximately 3.3 miles from the contraflow entry. This extension is shown in Figure 1. The cost to METRO and TSDHPT of implementing this additional corridor improvement was about $130,000, and all of it was from local sources.

In addition to the contraflow and concurrent-flow improvements to the North Freeway, METRO currently operates four park-and-ride lots (see Figure 1) in the North Freeway corridor. Transit service provides a total of 222 daily bus trips during the morning and evening operating periods. Likewise, the HOV facility serves approximately 350 authorized vanpools per operating period. Daily ridership for both buses and vanpools is currently 15,500 passenger trips daily.

To date there are three other contraflow operations nationwide: the Long Island Expressway in New York, the I-495 approach to the Lincoln Tunnel in New Jersey, and the US-101 Golden Gate approach to San Francisco. Of these facilities, only one is a joint local government project. The Lincoln Tunnel contraflow lane is a joint project between the Port Authority of New York and New Jersey and the New Jersey Department of Transportation.

SCOPE OF PAPER

The purpose of this paper is to discuss the joint operations management of the North Freeway contraflow lane project in Houston. In addition, the impact of this success on other interagency transportation-related efforts in Houston is presented.

First, the Operations Agreement and Operations Plan adopted by TSDHPT and METRO are outlined. These documents specify the responsibilities to be carried out by the two agencies. Responsibilities include the creation of a Contraflow Project Management Team that oversees the operations of the contraflow and concurrent-flow facilities. Second, a discussion of the matters faced by the Contraflow Project Management Team during the first 4 years is presented. Issues raised at the bimonthly meetings primarily involve law enforcement on the HOV lane, maintenance activities, operation policies, and physical modifications to the HOV lane that would improve HOV or adjacent freeway operation. Third, the impact that the successful management of this interagency project has had on other interagency transportation efforts in Houston is highlighted. Finally, a discussion is offered on the positive and negative aspects of an interagency approach to the development and operation of future HOV facilities.

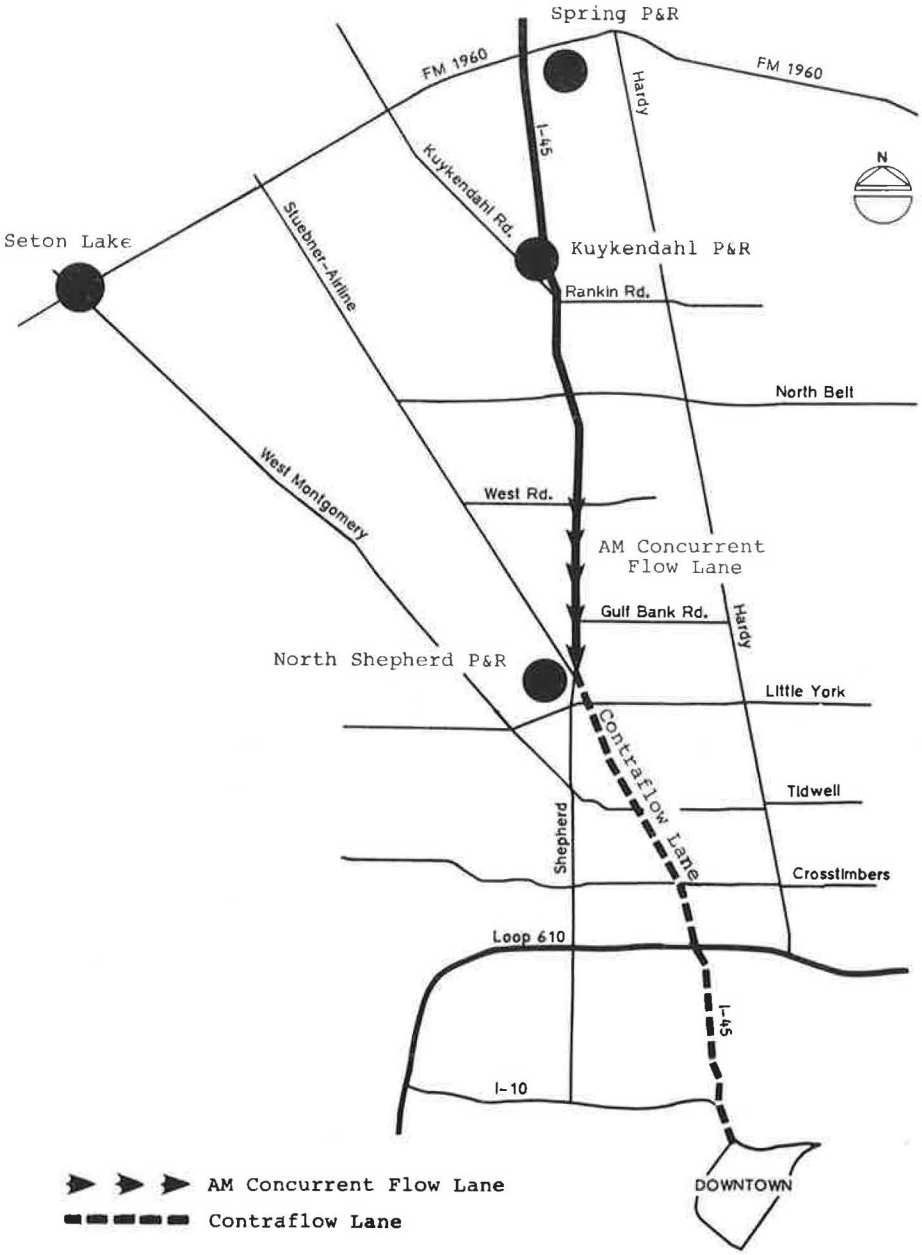

FIGURE 1 Contraflow and concurrent-flow lanes.

OPERATIONS AGREEMENT

The Operations Agreement defines the individual responsibilities that each agency has to the project. Deriving its focus from the expertise of each agency, surface and structural maintenance of the HOV facility is provided by TSDHPT, and the daily operation management of the facility is provided by METRO.

The predominant feature of the Operation Agreement was the establishment of a Contraflow Project Management Team that oversees all aspects of the HOV lane. Each agency was required to appoint one person who would serve as project manager. Bimonthly meetings are held to review operation policies and to address matters that require action or analysis.

All rules and regulations regarding use of the contraflow lane by authorized drivers are established and enforced by the Contraflow Project Management Team, as stated in the Operations Agreement. Regulations such as speed limits, individual qualifications for driver authorization, and vanpool ridership capacity requirements are jointly determined by METRO and TSDHPT. Any issues regarding policies or procedures are also channeled through the Team. The policy of having the project manager as the only recognized communication link was intended to minimize miscommunication between METRO and TSDHPT.

CONTRAFLOW MANAGEMENT ISSUES

The Contraflow Project Management Team meets on a bimonthly basis to oversee daily operational issues. The following discussion highlights some of the issues that have been raised at Team meetings. These are categorized as follows:

1. Operational modifications to the contraflow lane,

2. Maintenance of the contraflow and concurrent-flow facility, and
3. Carpools.

Operational Modifications to Contraflow Lane

Although the HOV facility was initially successful, the Contraflow Project Management Team did realize that certain operational modifications could further improve performance. Two notable operational modifications to the facility that were discussed at the Team meetings are briefly described.

Implementation of Concurrent-Flow Lane

During the planning and initial operations of the contraflow lane, traffic congestion in the morning peak started to extend several miles past the northern terminus. Consequently, the Team decided that it was necessary to consider means of extending priority treatments further north during morning operation. Unacceptable traffic conditions upstream of the existing limits prevented the borrowing of a lane for an extension of contraflow operations (1).

The Team decided that the median shoulder could be used as a concurrent-flow lane for the authorized contraflow users during the morning peak. The concurrent-flow lane could extend 3.3 miles. Median drainage inlet and median superelevations prevented any further extension.

TSDHPT subsequently designed necessary signing and striping modifications to convert the median shoulder for bus and vanpool use. Median pavement integrity was sufficient to support vehicle traffic. A connection ramp was designed at the downstream terminus to facilitate direct access from the concurrent-flow median shoulder to the entry of the contraflow lane. An exception was granted from Interstate standards by the FHWA in fall 1980 to use the median as a temporary lane during a 2.5-hr period each day. Project implementation was expedited by use of local monies from both agencies to fund construction. TSDHPT installed signs, restriped lanes to accommodate the lane over bridge decks, and reinforced bridge railing. METRO constructed a connection ramp and gate. The total cost to both agencies was about $130,000. Construction began in November 1980 and was completed about 4 months later.

As a measure of success of this facility, 90 percent of the contraflow ridership originates from the concurrent-flow lane. Travel time savings for users of the concurrent-flow lane range between 3 and 5 min.

Implementation of Simultaneous Setup Procedure During Contraflow Deployment

METRO has an 18-member crew that sets up and takes down the HOV facility in both the morning and afternoon hours of operation. Setup and takedown times following operation periods ranged from 1 to 1.5 hr.

This 1- to 1.5-hr transition time from mixed-flow operation to contraflow operation was perceived by the public as an indication of unsatisfactory use of the freeway. The congestion experienced by off-peak motorists further compounded the public's misconception. As a result of this negative public feedback, the Contraflow Project Management Team recognized the need to minimize the transition time.

The Team determined that construction of median operations north and south of the I-610 interchange would enable METRO's crews to set up and take down separate sections of the contraflow lane simultaneously. Figure 1 shows the relative proximity of I-610 interchange to the termini of the contraflow lane.

The $60,000 cost for the two median openings was shared by METRO and TSDHPT. It was incorporated into an existing construction project to rehabilitate the I-610 bridge structure. The simultaneous operation required no additional personnel or equipment.

The set-up and take-down times were essentially cut in half by the implementation of the simultaneous set-up and take-down procedure. Consequently, a reduction was achieved in the total time that the borrowing of a freeway lane from mix flow traffic is required.

Maintenance of Contraflow and Concurrent-Flow Facilities

Each agency's maintenance responsibilities for the contraflow lane were detailed in the Operation Agreement for the lane. These tasks were divided according to the strengths and resources of each agency.

Maintenance responsibilities of METRO to the contraflow lane included safety posts and holes, gates, beacons, lamps, pylons, service poles, control switches, signal controllers, and changeable message signs. These traffic control devices were installed as part of the original $2.1 million construction project. METRO's field crew(s) and electrical contractor are responsible for these tasks.

Maintenance of the roadway surface and strucures within the limits of the contraflow lane, which were not installed with the construction project, are the responsibilities of TSDHPT. In addition, maintenance of the needed signs and striping for the concurrent-flow lane are the responsibility of TSDHPT.

As METRO's staff is involved on a daily basis with the operation of the contraflow lane project, the Team meetings provide a needed forum for communication of maintenance-related issues to TSDHPT.

Carpools

The contraflow lane is reserved for the use of authorized buses and vanpools only. Persons advocating the use of carpools on the contraflow lane petitioned METRO to modify the existing authorization policy to include three-person carpools.

The Contraflow Project Management Team was requested to investigate the possibility of carpool use and make a recommendation to its feasibility. Local politics played a role in determining the final recommendation. By using existing operations data, the Team examined the issue from the viewpoint of (a) possible need for facility modification, (b) impact on enforcement, and (c) performance (i.e., increase of person trips versus increase of vehicles).

Analysis indicated that the existing facility would have to be modified with additional storage lanes and violator ramps. Unlike buses and vanpools, authorized carpools would be difficult to distinguish from nonauthorized passenger cars. Additional storage lanes for vehicle inspection would be required. The additional cost for the needed storage lanes and violator exit ramps would change substantially what was initially considered a low-cost capital project.

The number of persons who would be transported by carpools was considered nominal. It was determined that the extra capital cost and increased enforce-

ment outweighed the benefits of authorizing carpools to use the contraflow lane.

Based on the results of this analysis, the Team recommended to the respective administrations that carpools not be allowed to use the contraflow facility. The administration concurred with the Team's recommendations and upheld the carpool restriction.

IMPACT OF CONTRAFLOW PROJECT MANAGEMENTS' SUCCESS ON OTHER TRANSPORTATION-RELATED EFFORTS

The successful management of the North Freeway contraflow lane project created a unique spirit of cooperation between METRO and TSDHPT. This interagency cooperation formed the basis for other joint transportation-related endeavors:

1. Future HOV facility development,
2. Activities of a Houston Traffic Management Team, and
3. Overall system management of transportation facilities.

Future HOV Facility Development

Although the contraflow lane project has proved successful, it is only planned as an interim improvement to the North Freeway corridor. It was foreseen from the outset of the project that the borrowing of a lane from the off-peak direction would inevitably result in an unacceptable level of congestion in that direction. A study performed by the Texas Transportation Institute recommended that the contraflow lane not remain in operation past 1984, and that a median that physically separated the HOV lane be constructed in its place (2).

In addition, the yearly cost to METRO for the setup and takedown of the contraflow lane approaches is $600,000. Consequently, a facility that was not as labor intensive could significantly reduce the yearly operational expenses that were incurred. A barrier-protected HOV lane replacement could cut deployment costs by 75 percent or more.

For these reasons TSDHPT and METRO initiated efforts toward the development of the median transitway. TSDHPT also decided to incorporate programmed improvements to the freeway main lanes and parallel frontage roads as part of this one-time effort. Construction on the first phase of this replacement began in July 1983. The first section of the median busway should be completed by 1985. It is anticipated that overall construction along the North Freeway corridor will be completed in 1988.

The North Freeway project is only one of three transitway projects that are being jointly developed by the two agencies. As indicated by the data in Figure 2, by 1990 more than 40 miles of transitways will be constructed in medians of area freeways. This represents an investment from local, state, and federal sources of more than $150 million.

As indicated previously, all of these projects use the existing strengths of the two agencies. TSDHPT has the primary role of design supervision and construction management, whereas METRO has staff and resources to operate the facility and expedite development with local funding for design. Both agencies participate in the plan preparation process through joint project advisory teams. Representatives from the Houston Traffic and Transportation Department, the Houston/Galveston Area Council, the staff consultants, and the FHWA are members of the coordinating team.

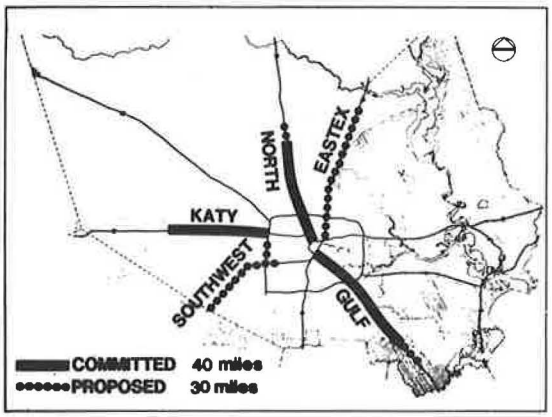

FIGURE 2 Committed and proposed transitways.

Activities of the Houston Traffic Management Team

Based on the success of the San Antonio corridor management team (3), it was apparent that the Contraflow Project Management Team provided a good start toward the creation of a similar team in Houston. The corridor, or traffic, management team represents an interagency approach toward solving transportation operational problems at the staff level.

The Houston Traffic Management Team is comprised of representatives from Harris County law enforcement agencies; city, county, and state transportation departments; and METRO. The Team meets monthly to discuss such topics as the review of traffic control strategies for major urban rehabilitation projects, review and approval of proposed operational changes to existing facilities, and operational problems encountered by law enforcement officials (4). An example of the Team's activities is presented in the following paragraphs.

The Team considered a proposal to restripe an extremely congested major thoroughfare from three 12-ft lanes in each direction to four 9-ft lanes in each direction. This arterial carries almost 80,000 vehicles per day. The Team reviewed a video tape of traffic flow on this arterial during peak periods. (This video tape was taken by one of the member agencies.) On review of this footage and consideration of the relatively insignificant level of bus and truck activity on this road, the Team agreed that the proposal, though radical, was appropriate. The involved agencies then took the steps necessary to accomplish the restriping. The Team continues to receive reports of the success of the restriping through reports from law enforcement and traffic engineering Team members. In addition, the Team favorably reviewed some time-lapse photography footage on this roadway after the narrow-lane operation was implemented. It was noted that bus and truck traffic was not adversely affecting traffic operations. (This footage was taken by another member agency.)

The most important result of the Team's activities since its inaugural meeting in January 1981 is the communication links that have been established between all transportation-related agencies within Harris County.

Regionwide Freeway Surveillance and Control System Development and Management

Part of the plan preparation for the transitway projects includes the design of the surveillance and control system for the operation of these facili-

ties. The control system to be used on all transitway projects would be comparable to those systems operating on freeways in Detroit and Los Angeles.

Although the initial installation of conduit and data-communication equipment is being accomplished primarily for the operation of the transitways, this equipment has the capacity to be used for freeway and arterial traffic management.

Some of the design work for transitway surveillance and control is being performed by TSDHPT. This represents another example of the existing strength of an agency being utilized. In this case the Department's Automation Division, which has developed computerized traffic signal systems within Texas, is developing the software for the system. TSDHPT anticipates expanding on the base system that will be installed through the transitway projects. Consequently, the potential will exist for the joint operational management of the corridor.

CONCLUSION

METRO and TSDHPT have successfully managed the North Freeway contraflow lane project. This joint management approach has several advantages.

1. The resources of each agency are effectively used. METRO provides the operational support, whereas TSDHPT provides roadway maintenance.
2. Limited economic resources can be combined to finance capital projects. The North Freeway transitway project, which will replace the existing contraflow facility, is being financed with funds from FHWA, UMTA, TSDHPT, and METRO. The total project cost is $130 million, which includes improvements to freeway and frontage roads.
3. Joint agency projects enable the transportation needs of a corridor to be more effectively addressed. As part of the contraflow lane project TSDHPT installed ramp metering signals at all entrance ramps within the contraflow and concurrent-flow lane to improve mixed-flow traffic movement and help minimize the negative impacts of contraflow to main-line traffic.

Joint agency management of a project is not without its shortcomings. Joint projects require review and approval by each participating agency. This increase of bureaucratic review requires greater lead time to develop and implement a project.

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Abridgment

Delay Messages and Delay Tolerance at Houston Work Zones

R. DALE HUCHINGSON, JOHN R. WHALEY, and NADA D. HUDDLESTON

ABSTRACT

A questionnaire survey of freeway drivers in Houston was conducted at six locations: I-10 W, I-10 E, I-45 N, I-45 S, US-59 S, and US-59 N. The questionnaire data were based on a license-plate survey with 843 driver responses to a mail-in questionnaire. The questions were related to two issues. Specifically, the survey sought to determine (a) the minutes of delay connoted by the expressions Major Delay and Minor Delay, and (b) drivers delay tolerance, that is, the minutes of delay before a driver would divert from a freeway to a service road. The reason for studying drivers' responses on three freeways and at six locations was to assess possible regional differences among Houston drivers based on driving experiences unique to the facility. The questionnaire data found no significant differences among the freeway drivers at the six sampled locations. The message Major Delay uniformly meant 22.7 min or more delay time, with a range of 21 to 25 min across locations. The message Minor Delay meant 7.9 min or less delay time, with a range of 7 to 9 min across locations. These time values were largely similar to a previous national survey of the meanings of major and minor as adjectives describing accident severity. Regarding delay tolerance, Houston drivers stated they would divert from the freeway to a service road given news of a delay of 5 or 6 min. Delay tolerance was 10 min less for the Houston sample than for previously researched national samples. Possible reasons for the reduced delay tolerance are discussed.

In previous research by the Texas Transportation Institute (TTI) (1), a series of questionnaire surveys were conducted on the topic of drivers' expressed delay tolerance to various delay periods. One study was conducted in each of three geographical regions--College Station, Los Angeles, and St. Paul. One question asked whether or not the driver would stay on the freeway or divert on reaching various indicated periods of delay: 5, 10, 15, 20, or 30 min; or 1 or 2 hr.

The responses across regions were found to be remarkably consistent. For each regional population, approximately two-thirds of the respondents stated that they would divert on reaching a 20-min delay. The median value was within 15 to 20 min for all groups. There were some differences at the upper end of the distribution. For example, 95 percent of the California respondents would divert at a 30-min delay in comparison to 85 percent of the St. Paul sample and 80 percent of the College Station sample. Almost everyone would divert with an hour's delay, but there was a hard-core group who would not divert under any circumstances.

The study suggested several types of incidents causing the delay: roadwork, accident, truck overturned, ice, and rain. The type of incident had no effect on the median diversion delay (15 to 20 min). The weather appeared to be a less cogent reason than accident or roadwork, and more drivers refused to divert for these reasons at delay periods up to an hour. Time periods of 1 and 2 hr were synonymous for purposes of diversion. Anyone who would divert at all would divert to an hour delay.

Another study in the temporal factors series examined the meaning of the expressions Major Accident and Minor Accident. If these expressions could be used instead of the exact minutes of delay, the operating agency would be relieved of the problem of continually updating the message for accuracy and the associated problem of message credibility. Drivers in the major group were asked to indicate the minimum number of minutes delay implied (X minutes or more). Drivers in the minor group were asked to give the maximum delay implied (X minutes or less).

It was found that minor implied 12 min or less to the median driver, whereas major implied about 22 min or more. Although the study did not yield a best estimate of the exact minutes of delay implied, it did bracket the upper limit of the minor group and the lower limit of the major group. Delay periods of between 12 and 22 min would not aptly be described by either expression.

In the study conducted in Houston drivers were not asked if they would divert in response to the message. However, generalizations could be made from the first study. Because Major Accident implied at least a 22-min delay, and study 1 found that two-thirds of the respondents stated that they would divert with a 20-min delay, it was deduced that at least two-thirds would also say that they would divert in response to the major message. Similarly, less than 20 percent would divert with a 10-min delay. Hence, few could be expected to divert in response to the minor message.

An objective in the present study related to the meaning of Major Delay and Minor Delay rather than to the word accident. Another temporal factor study in the earlier research found that the word delay means many different things to drivers, but the most popular meanings were that freeway travel will be so-many minutes longer than usual, or that one will arrive so-many minutes later than usual. It did not necessarily mean traffic was stopped that long or that it would take that long to remove an obstruction.

It was concluded that temporal information in terms of minutes delay is an effective method of traffic control when the objective is to induce diversion of various percentages of the drivers.

In the Houston questionnaire research, TTI explored further the meaning of the words major and minor as modifiers of the word delay. A sign was said to be just ahead of a highway construction work zone. The messages were Roadwork Ahead followed by either Minor Delay or Major Delay. The seven multiple-choice alternatives were identical to those in

the earlier research. The meaning of delay was clarified also by asking: "How long would you expect to be delayed in comparison with your normal travel time?" This is the most common meaning of delay, but in this research it was defined as being in comparison to some individual travel time.

To determine delay tolerance, the driver was told there was a service road next to the freeway and was asked how long he or she would wait to exit to the service road. The same seven delay durations were given. However, a final option was "I would not leave the freeway."

A unique feature of the research was that drivers were classified in terms of the particular Houston freeway on which they were driving at the time their license plate was noted. It was assumed that there might be regional socioeconomic differences in the samples that would influence their responses. In addition, there could be other unique driving experiences. If no differences were found, this would lend further support to the generality of the findings.

METHOD

The research technique involved administration of a one-page mail-in questionnaire to a sample of 843 Houston freeway drivers who responded to a TTI letter. The survey dealt with their interpretation of the modifiers of the word delay and the duration of delay they would tolerate before exiting to a service road.

Subjects

Respondents were recruited from a license-plate study. The respondents were observed driving on one of three Houston freeways (I-10, I-45, or US-59). Both directions of travel were observed on each freeway. Hence, there was a set of six driving populations: I-10 W, I-10 E, I-45 N, I-45 S, US-59 S, and US-59 N.

Procedure

Houston motorists were surveyed at the six locations on the outside of Loop 610. License-plate data were collected to obtain mailing addresses used for mailing the questionnaires. The survey was conducted twice. It was conducted once during peak hours in July and again in August 1981. From 537 to 638 license-plate numbers were recorded for each of the six freeway conditions. To ensure correct identity of the respondent, the questionnaire forms were color coded.

Of the 3,543 letters sent out, 843 (23.8 percent) were completed and returned. The numbers and percentages of responses varied among the six conditions, but the response percentage ranged from 19 to 31 percent.

The letter asked the motorist to help TTI evaluate the meaning of messages that might be displayed on signs in advance of freeway construction and maintenance operations. The questionnaire consisted of only four questions. These questions covered the following items:

1. Freeway driven most often,
2. Major delay message meaning (check one of seven: 5, 10, 15, 20, or 30 min; or 1 or 2 hr),
3. Minor delay message meaning (check one of seven by using the same range of times), and
4. Amount of time one would wait to exit to a service road (check one of eight by using the same times plus "I would not leave the freeway").

RESULTS AND DISCUSSION

Interpretation of Major and Minor Delay

The data in Figure 1 and Table 1 summarize the responses to the different questions for the six conditions. The arrows on Table 1 indicate that the median or 50th percentile response would fall between the two time delays shown on the left. It may

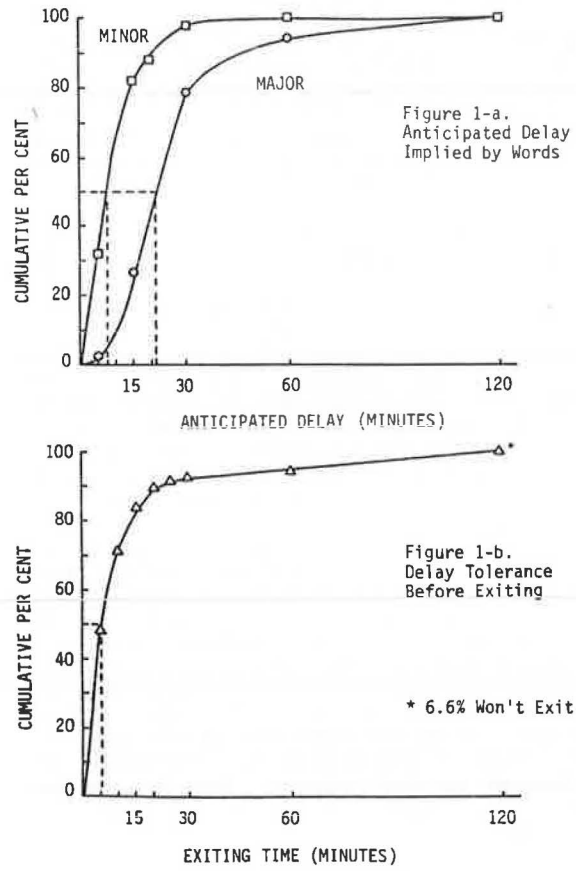

FIGURE 1 Delay interpretation and tolerance.

TABLE 1 Interpretation and Tolerance

DELAY	INTERPRETATION						TOLERANCE		
	Major Delay			Minor Delay			Exiting the Freeway		
	N	%	Cum %	N	%	Cum %	N	%	Cum %
5 Min.	20	2.63	2.63	244	32.11	32.11	361	47.50	47.50
10 Min.	53	6.97	9.60	233	30.66	62.77	180	23.68	71.18
15 Min.	129	16.97	26.57	145	19.08	81.85	100	13.16	84.34
20 Min.	119	15.66	42.23	42	5.53	87.38	39	5.13	89.47
30 Min.	278	36.58	78.81	78	10.26	97.64	23	3.03	92.50
1 Hour	122	16.06	94.87	16	2.11	99.75	6	0.79	93.29
2 Hours	39	5.13	100.00	2	0.25	100.00	1	0.13	93.42
Won't Exit	---	-----	------	---	-----	------	50	6.58	100.00
TOTAL	760	100.00	------	760	100.00	------	760	100.00	------

be noted that Major Delay fell between 20 and 30 min and that Minor Delay fell between 5 and 10 min at all six locations. Regardless of the driving population, the interpretation was standard.

By interpolation, it was possible to make a point estimate of the minutes of delay associated with the 50th percentile point. These estimates are summarized in Table 2. Note that at all freeway locations Major Delay meant at least 21 min to at least 25 min and Minor Delay meant no more than 7 min to no more than 9 min. There were no differences in driver interpretation at any of the study locations ($\chi^2 = 0.3$; df = 5). The average driver on all freeways described major delay as 22.7 min or more and minor delay as 7.9 min or less.

TABLE 2 Median Delay Interpretation and Tolerance

	Delay Interpretation		Delay Tolerance	
Freeway Location	Major Delay (min)	Minor Delay (min)	Exit Freeway (min)	Won't Exit (%)
I-10 W	21.0	7.0	4.0	4.5
I-10 E	23.0	9.0	6.0	10.7
I-45 S	22.0	7.5	5.0	9.2
I-45 N	22.0	8.0	6.0	7.1
US-59 S	23.0	8.5	7.5	5.9
US-59 N	25.0	7.5	5.0	2.2
Mean	22.7	7.9	5.6	6.6

Delay Tolerance

The findings regarding minutes of delay before a driver would exit to a service road are also shown in Figure 1 and Table 1. For all except one freeway location, the median or 50th percentile point fell between 5 and 10 min.

The data in Table 2 also summarize point estimates by interpolation for the delay tolerance data. For I-10 W, the 50th percentile was slightly less than 5 min. For the other freeway conditions, the 50th percentile point was slightly more than 5 min. The delay estimates were not significantly different among the six freeway locations ($\chi^2 = 1.1$; df = 5). The average driver on each freeway had a delay tolerance of about 5.6 min. About 6.6 percent of all drivers indicated they would not divert to any delay message.

Discussion of Results

The findings of this study supported those of previous research with respect to the meaning of Major Delay. In the earlier study Major Accident implied 22 min or more delay. Here, Major Delay also implied 22.7 min or more delay. Previous research found that Minor Accident meant about 12 min or less delay, whereas the present study found that Minor Delay meant 7.9 min or less delay.

Nevertheless, delay tolerance was radically different for the Houston sample. Whereas the earlier sample (College Station, Los Angeles, and St. Paul) would tolerate 15 to 20 min delay, the Houston sample was prepared to exit to a service road in 5 to 6 min. There are several possible reasons for the difference.

1. In the Houston study a convenient service road was posed. But in the earlier study the alternatives were to either "stay on" or "get off" with no suggestion as to alternate facilities. The continuous frontage roads in the Houston region provide convenient alternative routes that are not usually available in other regions of the country.
2. The Houston sample was known to be of all experienced freeway drivers. However, in the previous study the sample was not recruited from freeways and may have been less experienced in knowing what they might do in such a situation.
3. Construction work is known to involve a prolonged duration, whereas some of the incidents in the earlier research could clear up more quickly (e.g., by removing a wrecked vehicle from the roadway).

CONCLUSIONS

The results of a questionnaire survey conducted in Houston failed to find any difference between drivers on three freeways (either direction) in terms of their interpretation of work-zone messages of Major Delay or Minor Delay. The survey also found the six groups were consistent in how long they would wait before exiting to a service road.

The findings largely support previous research (1) with respect to the meaning of the word major (22 min or more) or minor (about 8 min or less). The Houston research, in comparison to previous research (1), reported a much more brief delay tolerance (5 to 6 min) when a convenient service road existed as an alternate route.

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Optimizing Traffic Diversion Around Bottlenecks

YI-CHIN HU and PAUL SCHONFELD

ABSTRACT

A traffic simulation and optimization model has been developed to analyze traffic flow in large networks with severe queuing. The model can be used to evaluate the impacts (e.g., travel time, operating costs, accidents, fuel consumption, and pollutant emissions) of any assignment over time and to minimize combinations of such impacts. The influences on optimal assignment of traffic inflow rates and durations, relative route lengths and capacities, queue storage capacities, and other factors are shown for a simple network. A comparative analysis of route-diversion and capacity-expansion alternatives is given for the more complex network on Maryland's Eastern Shore.

A bottleneck along a highway may be defined as a relatively short section with substantially less capacity than the rest of the road. For example, a temporary bottleneck may be caused by road repair work or an accident, whereas a long-lasting bottleneck may be caused by any narrowing of the road (e.g., lane drops, narrow bridges) or interruptions in flow (e.g., at intersection, railway crossings, drawbridges). Whatever the reason for the bottleneck, and however short it is, its capacity sets the limit for the total roadway capacity and degrades service levels even before that capacity limit is reached.

When roadway volume v exceeds bottleneck capacity C_b, a queue would grow upstream of the bottleneck at the rate R, where

$$R = v - C_b \tag{1}$$

Hence the queue length at time t is

$$\ell(t) = \int_0^t R(t) \tag{2}$$

If alternate routes around the bottlenecks are available and used by motorists when volumes are high, the queuing delay costs may be considerably reduced. Such route diversion may occur spontaneously. In theory (1), unrestricted motorists would choose their routes in such a way as to equalize travel impedance along all alternate paths that are actually used. That theory presupposes that motorists have perfect information and make optimized rational decisions. In practice, motorists may often have little information on which to base a rational decision, especially if they are unfamiliar with alternate routes or if a temporary bottleneck develops downstream. Furthermore, in most networks the user-optimized traffic assignment results in higher total impedance than a system-optimized assignment determined by a central controller. Hence at locations where alternate routes are available around bottlenecks it is desirable to have either a set of route-diversion guidelines prepared in advance or an algorithm that can determine the optimal assignment. In either case it should be possible to specify the fraction of traffic to be assigned to various routes at various times.

For this purpose, a macroscopic traffic simulation and optimization (TSAO) model was developed in a recent study (2-4). This model can simulate queuing upstream from bottlenecks [including the interactions of queues from various network links by using Lighthill's shockwave function (5)]. The model can predict traffic impacts and determine the assignment that minimizes a specific objective function (e.g., travel time or total system costs). Event-scan time management was used to enhance the computation efficiency in large network applications. A detailed description of this model is available elsewhere (4).

Although this model can deal with large highway networks and demand distributions that are complex over both space and time, it is primarily used here to examine basic parameters (e.g., traffic inflow rate and duration, length and capacity of alternate routes, relative locations of bottlenecks and diversion points) that influence the optimal diversion strategy in a simple network, which is shown in Figure 1. Although this application grossly underuses the capabilities of the model, it permits researchers to obtain simple traffic diversion guidelines that are applicable at many locations with similar network configurations. A similar analysis and set of guidelines might also be developed for a family of simple network configurations (e.g., with more alternate routes and varying locations of diversion and reentry points). However, as network complexity increases, it becomes less practical to specify general diversion guidelines and more desirable to simulate the actual network with its specific characteristics. Some results of such an application for the Maryland Eastern Shore network are given later in this paper.

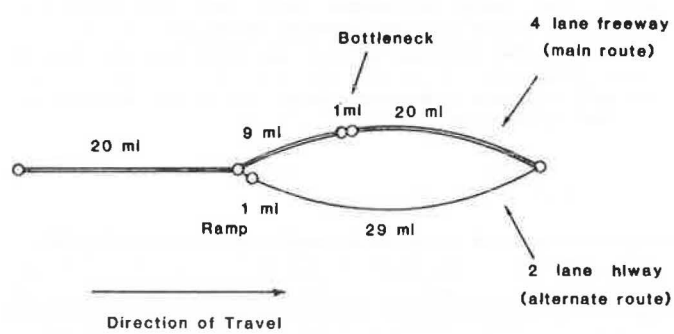

FIGURE 1 Base network.

SIMPLE NETWORK

The baseline network shown in Figure 1 consists of a main route, which is a four-lane freeway, except for a short bottleneck, and a two-lane alternate route. The bottleneck is a section of two-lane rural highway, 1 mile long, with a baseline capacity of 900 vehicles per hour in the relevant direction. The capacity of the alternate route (750 vehicles per hour) is controlled by the entry ramp from the main route at the diversion point. The two routes are equal in length (30 miles between the diverge and merge points), and the distance between the di-

version point and the bottleneck on the main route is 9 miles. In the baseline the traffic inflow rate equals 125 percent of the total corridor capacity, and an inflow duration of 2 hr is used.

Two cases were considered for the alternate route:

1. No opposing traffic is considered downstream of the ramp, which means that vehicles can travel faster than on the ramp; and
2. Heavy opposing traffic has limited the available capacity on the whole stretch of the alternate route, so that the downstream section has the same capacity as the ramp.

Because of the opposing traffic, case 2 provides a much lower level of service on the alternate route. The free-flow travel time on the alternate route is the same as on the main route in case 1 and is 50 percent longer than on the main route in the second case.

The objective function to be optimized is a total cost function that consists of the value of users' time, vehicle operating costs, and accident costs. An occupancy of two passengers per vehicle and a value of time of $4 per passenger hour ($8 per vehicle hour) were used. The consumer price indices of December 1982 were used to update the vehicle operating costs and accident costs from the 1977 AASHTO manual (6).

Sensitivity analysis (2) has demonstrated that the optimal diversion percentage does not depend substantially on values of travel time, car occupancy rates, or gasoline prices, and hence these are omitted from the factors considered in the following subsections.

Inflow Duration

Figures 2 and 3 show the optimal percentage of traffic taking the main route for various durations of inflow and for both cases. The following observations are made.

1. The optimal fraction of traffic desirable on the main route decreases sharply when the volume capacity ratio in the corridor increases from 0.50 to 1.0. In the first case inflow duration does not affect the optimal assignment if the corridor capacity is not exceeded. In oversaturated conditions a slight overassignment to the main route turns out to be optimal. Inflow duration has a small effect: the longer the duration, the closer the optimal as-

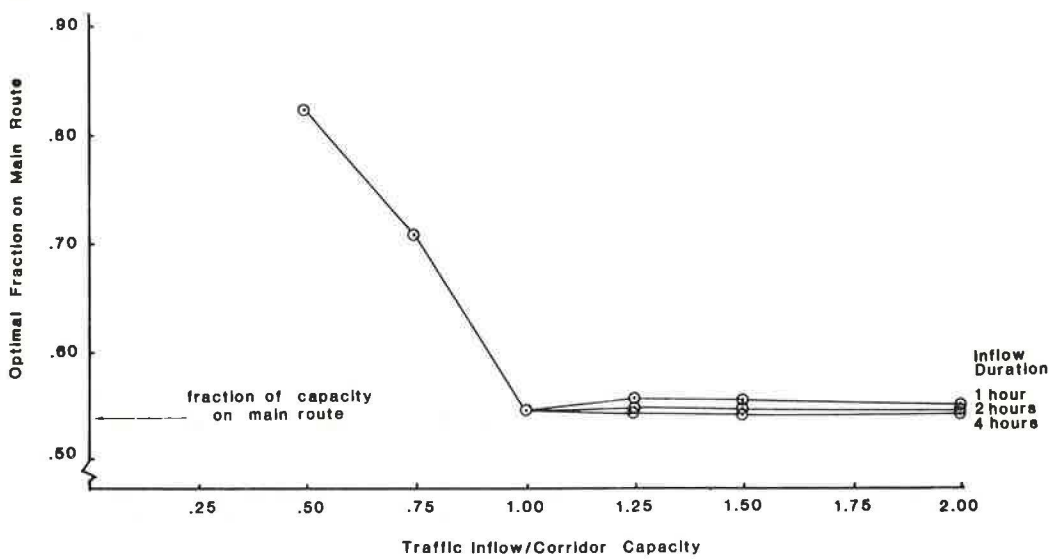

FIGURE 2 Effects of inflow duration on optimal assignment (case 1).

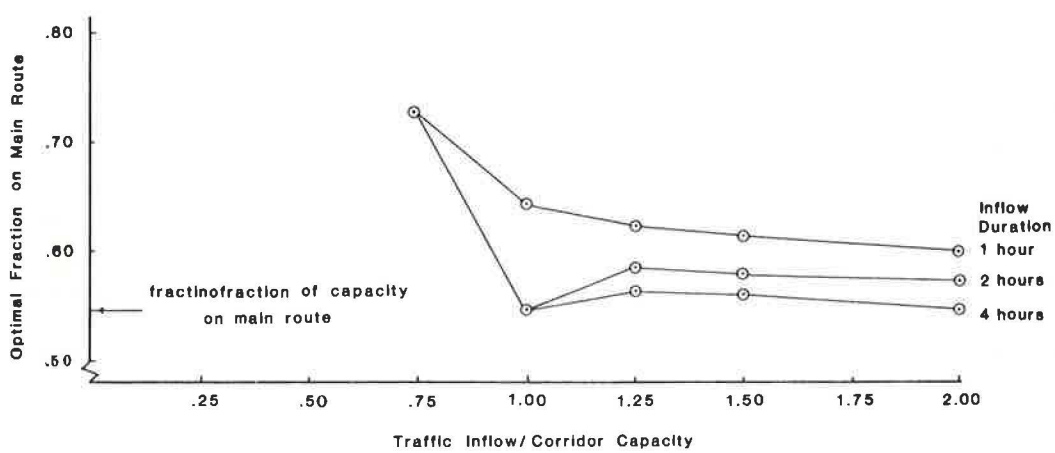

FIGURE 3 Effects of inflow duration on optimal assignment (case 2).

signment fractions approach the capacity fractions, as shown in Figure 2.

2. In case 2, where the overall service level on the alternate route is less than in case 1, inflow duration affects the optimal assignment more significantly. If the inflow rate equals the corridor capacity and if the duration is as short as 1 hr, the optimal assignment will allow a queue to develop on the main route. As the duration increases, the no-queue assignment becomes optimal, as shown in Figure 3. In oversaturated conditions, where traffic inflow rate exceeds corridor capacity, a small overassignment to the main route is preferable. The desirable degree of overassignment decreases as inflow duration lengthens (as in case 1) but to a larger extent, especially when the demand peak is short.

Travel Time as the Objective Function

If travel time is the only impact to be minimized, the optimal assignments may change slightly. Figures 4 and 5 show the minimum time assignments for cases 1 and 2, respectively. In general, travel time minimization favors more diversion to the alternate route than cost minimization, especially in undersaturated conditions. If the corridor capacity is exceeded, the differences between minimum time and minimum cost assignments become negligible.

Network Configuration

The relative route lengths and bottleneck locations affect optimal assignments, especially insofar as the lengths of queue storage sections are changed. Figure 6 shows that as the length of the alternate route increases, the fraction of the traffic staying on the main route should increase. The effects of length variations are significantly larger in case 2, where the traffic in the opposite direction downgrades the overall level of service on the alternate route. If the alternate route length is doubled, the optimal fraction on the main route increases from 0.58 to 0.72 in case 2, and from 0.55 to 0.64 in case 1.

If there is adequate queue storage area on the main route, it may be preferable to allow a queue to develop there. Figure 7 shows how the length of the storage section affects the optimal assignments. In the first case, where the alternate free-flow time on the alternate is as satisfactory as on the main route, the optimal assignment is insensitive to the location of the bottleneck. In case 2 the optimal assignment increases the fraction on the main route if the length of the storage section is less than 3 miles. Beyond 3 miles, for the given inflow rate and duration, any increase in storage will not change the optimal assignment significantly.

Variations in Capacities

If the capacity of the bottleneck or of the alternate route is expanded, a redistribution of traffic should occur to minimize system costs. In Figure 8 the 45-degree line indicates assignment ratios equal

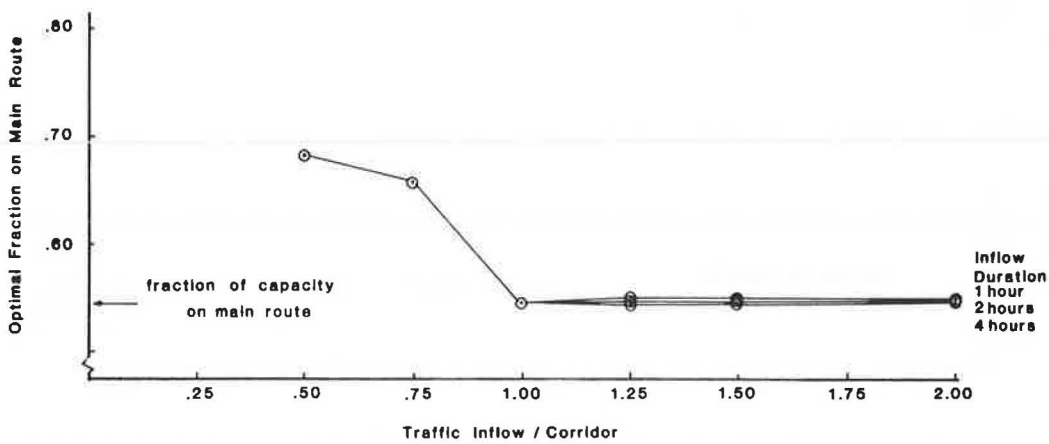

FIGURE 4 Optimal fraction on main route for minimizing total time (case 1).

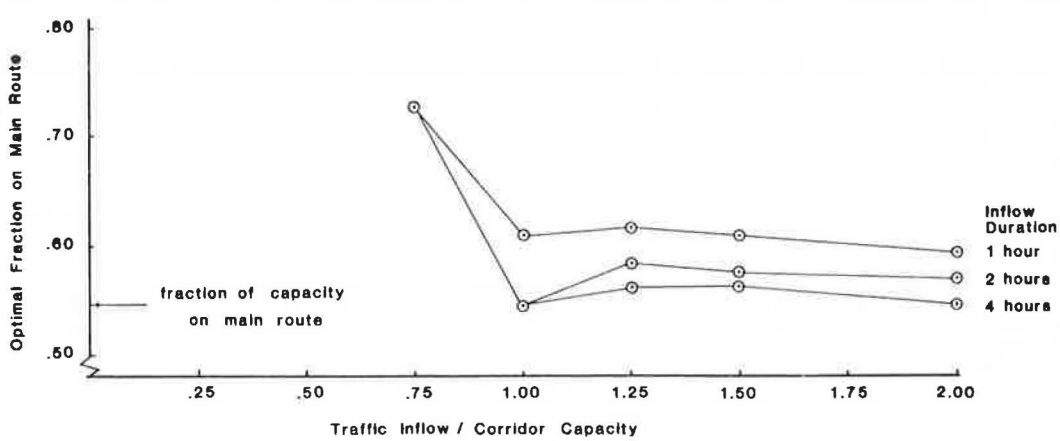

FIGURE 5 Optimal fraction on main route for minimizing total time (case 2).

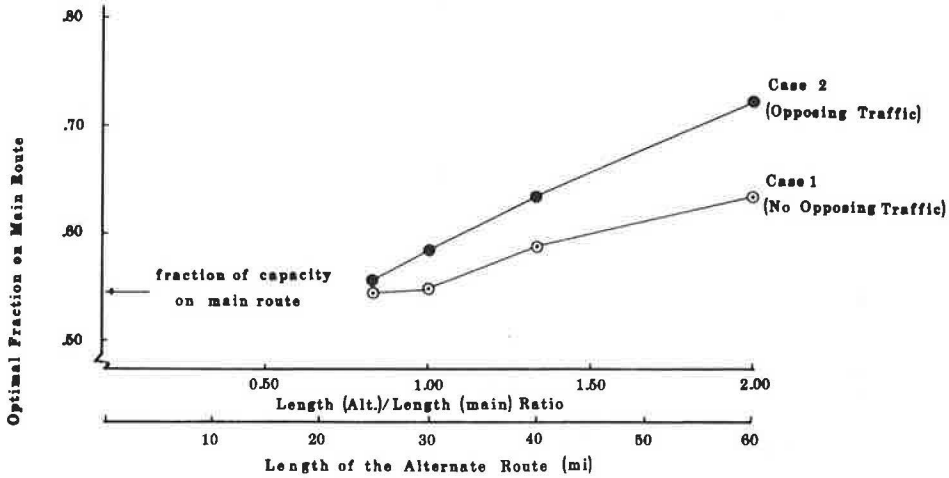

FIGURE 6 Effects of length ratio on optimal assignment.

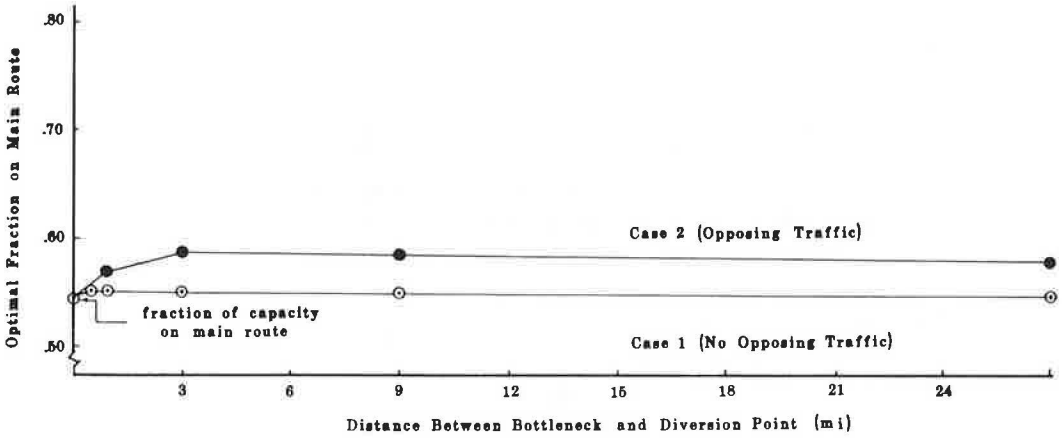

FIGURE 7 Effects of location of bottleneck on optimal assignment.

to the route capacity ratios. An optimal assignment above this line means an overassignment to the main route.

As shown in Figure 8, the lines obtained by connecting the optimal assignments for various capacity distributions are above and nearly parallel to the 45-degree line in both cases. This means that traffic should be slightly overassigned to the main route, but the degree of overassignment is insensitive to the capacity ratios. However, the degree of overassignment is larger in case 2 than in case 1.

COMPLEX NETWORK

In a recent study (2) the TSAO model was applied to the Maryland Eastern Shore network, which is shown in Figure 9. Summer recreational traffic between the Atlantic Ocean resorts and the metropolitan areas of Baltimore and Washington, D.C., creates severe congestion on the network, especially at two narrow bridges at Cambridge and Vienna. Capacity-expansion projects can improve the level of service, but the demand peaks occur too infrequently (15 summer weekends in this case) to justify any large-scale construction. Hence the TSAO model was used to analyze various alternatives for improving the quality of service on the network, including bridge reconstruction, lane widening, and route diversion.

The network consists of 30 nodes and 35 links and covers an area of approximately 3,000 miles^2. Traffic between eight origin-destination pairs, including divertible through traffic and nondivertible local traffic, was simulated, and the demands were expressed as time-varying step functions. Periods of up to 15 hr of traffic had to be simulated.

The following alternatives for improvement were analyzed:

1. Do-nothing;
2. Reconstruction and widening of Cambridge and Vienna bridges into four-lane bridges;
3. Additional left-turn lane on US-50 at MD-404;
4. Avoiding stop signs at the junction of MD-313 and MD-14 at Eldorado and MD-313 and MD-54 at Mardela Springs by providing right-turn ramps and acceleration lanes;
5. Additional lane in each direction on US-50 from the end of the Bay Bridge to Wye Mills;
6. Additional lane on MD-404 from Wye Mills to the junction of MD-16;
7. Combination of alternatives 2, 3, and 5;
8. Combination of alternatives 3, 4, and 5;
9. Combination of alternatives 6 and 8;
10. Optimal route diversion;
11. Combination of alternatives 3, 4, and 10;
12. Combination of alternatives 7 and 10;
13. Combination of alternatives 8 and 10; and
14. Combination of alternatives 9 and 10.

The present and future costs of various alternatives were determined by applying the model to the

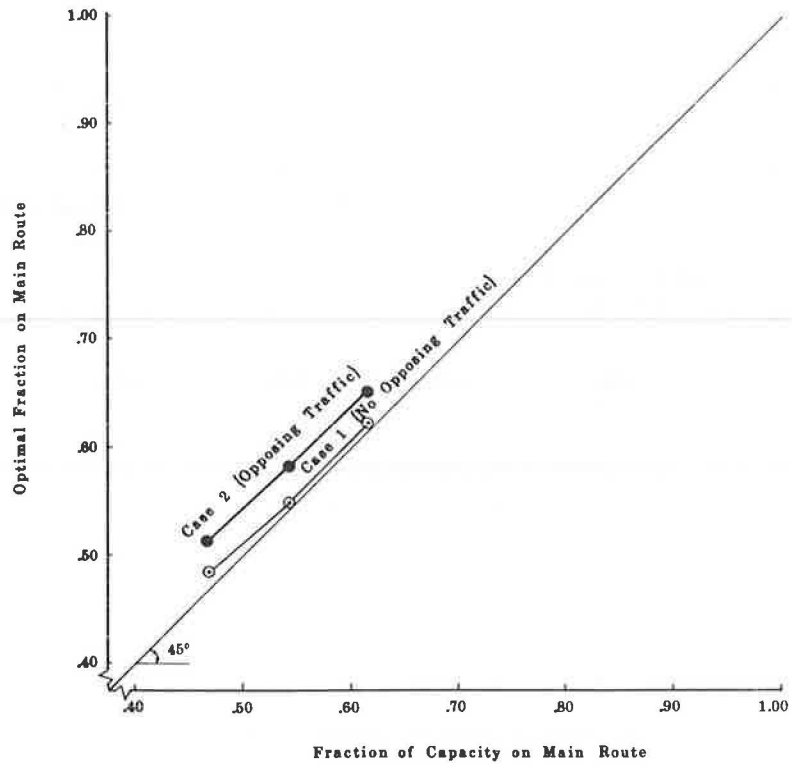

FIGURE 8 Effects of capacity ratio on optimal assignment.

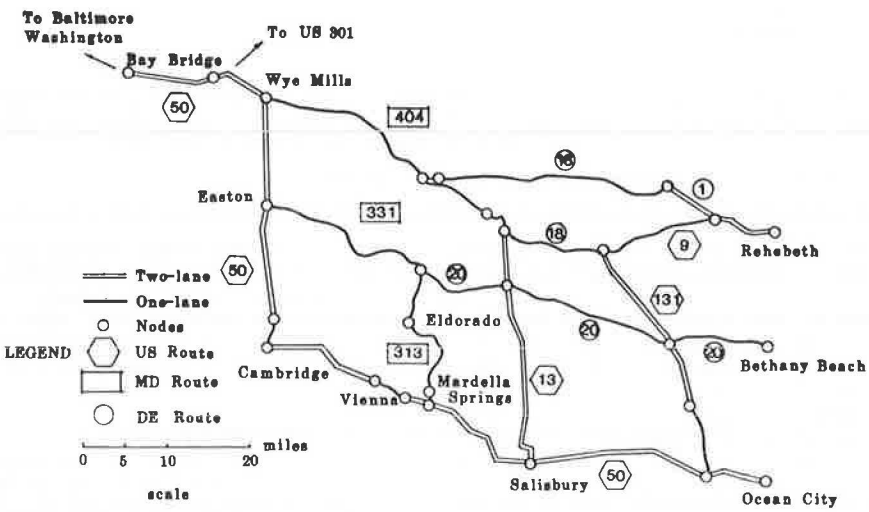

FIGURE 9 Eastern Shore network.

appropriate network configurations and traffic projections. An investment analysis was performed to determine the optimal timing of the improvements. Figure 10 shows the results of the investment analysis. The alternative with the largest equivalent uniform annual net benefit is preferable. The best two alternatives--13 and 14--incorporate route diversion and construction projects. The worst alternative is number 2, which involves bridge capacity expansion only; it has negative net benefits.

The analysis (2) tested the sensitivity of the results to parameters such as interest rate and value of time, and no significant change was found. The rankings of the alternatives were not significantly changed and the cost-effectiveness of route diversion was not lost, even when it was assumed that motorist inflexibility prevented optimal assignment and diminished the achievable benefits by as much as 75 percent.

CONCLUSIONS

If traffic inflow exceeds total corridor capacity, queues develop upstream of bottlenecks. The system-optimized flow pattern depends on the following factors:

1. Traffic inflow rates and peaking patterns,
2. Capacities on the main and alternate routes,
3. Duration of inflow exceeding capacity,
4. Lengths of the main and alternate routes,

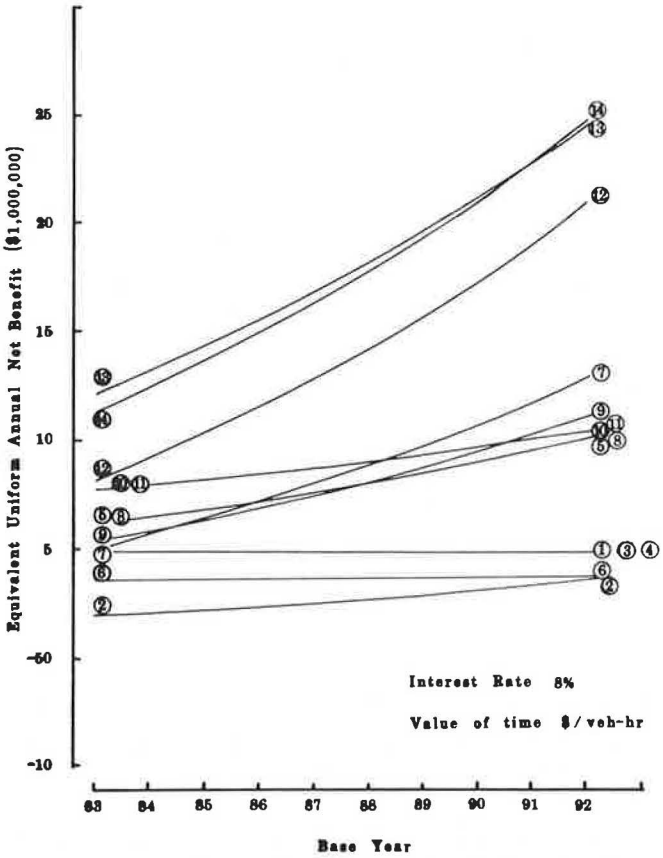

FIGURE 10 Equivalent uniform annual net benefit for various alternatives (base case).

5. Relative locations of the bottleneck and diversion point, and
6. The volume of traffic in the opposite direction on the alternate route.

Generally, it is preferable to assign slightly more traffic to the main route than its capacity share in the corridor, even if the lengths on the main and alternate routes are equal. The degree of overassignment increases with the length of the alternate route and the traffic volume in the opposite direction. If the duration of excessive inflow is short, it is desirable to allow queuing on the main route if adequate storage is available between the diversion point and the bottleneck. As the peak period lengthens, reduced overassignment to the main route is desirable. The effects of queue storage length and inflow duration become more significant as the travel time on the alternate route lengthens the increase.

The problem of determining the optimal assignment becomes more complicated if (a) time-varying traffic demand is considered, (b) more complex networks with many diversion points are studied, and (c) local traffic is considered.

The TSAO model is applicable to complex traffic flow optimization problems. It can be used to determine the timing and extent of a diversion and to provide information for evaluating and programming improvements in a network.

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Abridgment

Assessment of Violations on Priority Entry Ramps

JOHN M. MOUNCE

ABSTRACT

Priority entry ramps are a transportation system management technique implemented to provide travel time savings to high-occupancy vehicles delayed in access to freeway main lanes because of congestion. The effectiveness of priority entry ramps is diminished as violations by unauthorized, low-occupancy vehicles increase. Violations on priority entry ramps were analyzed from data collected at two sites in Houston. The results indicate an approximate, average violation rate that exceeds 40 percent when no enforcement is present. The results of random, intermittent enforcement indicate a decrease in violations to an approximate average rate of 15 percent. The priority entry exposure ratio, which is defined as service time of nonpriority vehicles divided by the arrival time headway between priority vehicles, was also determined to be an influencing factor on violation rate. For the limited data within this study, a linear equation was statistically established to predict a decrease in ramp violations as the priority entry exposure ratio increases. A graphical illustration of this relationship is presented to allow determination of enforcement requirements for priority entry ramps.

Priority entry ramps are specially designated or physically separated preferential lanes that allow high-occupancy vehicles (HOVs) to bypass low-occupancy vehicles delayed in access to freeway main lanes because of congestion. This transportation system management (TSM) technique is implemented to produce travel time savings as an incentive for motorists to use an HOV mode of travel and, therefore, to increase the person capacity of freeway corridors. National statistics on priority entry ramps report approximately 200 installations either in operation or planned (1).

The efficiency or effectiveness of priority entry ramps is diminished as violations by unauthorized vehicles increase. The violation rate or ratio, which is the total number of unauthorized vehicles using the priority ramp divided by the total number of vehicles on the priority ramp, is influenced by various factors (2). A wide range of violation rates on priority entry ramps (bypass ramps) have been observed--from 0 to 40 percent (3). Other studies (4) have purported a mushrooming effect of violation rate if enforcement is not observed at priority entry ramps. The objectives of this study were to determine the level, influencing factors, and potential alleviation or control of violations on priority entry ramps from study data at sites in Houston.

SITE DESCRIPTION

Priority entry ramps were installed in Houston at two sites on the Southwest Freeway (US-59 S) more than 5 years ago. The locations were along the Southwest Freeway (US-59) at Bellaire Boulevard and Hillcroft Avenue. Each of these sites has ramp metering signalization for nonpriority traffic. The priority and nonpriority ramps at each site are physically separated at both the connections to the frontage road and the freeway merge points. Ramp alignment at both sites is similar. Buses and vanpools are authorized to use the priority entry ramps with access at all times. Significant travel time savings are realized by priority vehicles because of the queuing of nonpriority vehicles at the ramp signal. Enforcement is applied on a random basis to control the violations to both signals and priority ramp use.

STUDY DESIGN

Both operational performance data and physical inventory data were collected at each of the two priority entry ramp sites in Houston. Each location was monitored over an extended period (October to December 1982) during typical weekdays (Tuesday through Thursday) during the morning peak period in clear and dry weather with no incidents. Distinctions were made within the data set as to the presence or absence of enforcement. All data were measured manually from an inconspicuous observation point so as not to be perceived as enforcement personnel and thus influence violations.

Operational performance data were separated by measured conditions on the priority ramp versus the nonpriority ramp. A preliminary examination of this data set indicated an apparent relationship among nonpriority service rate (time waiting in queue), priority vehicle arrival rate (time headway between vehicles), and violation rate. This led to a subsequent data-collection effort at the previous two sites over a 1-month period (March through April 1983) with no enforcement influence.

Consideration was also given to the nature and impact of priority entry violations on safety. Accident records at the two ramp sites were examined for a 5-year period. No accident was found at either site that could be attributed to or result from a violation of the priority entry ramp. It was observed during the field data-collection effort, however, that there were several instances associated with violation maneuvers that caused operational conflicts or potential conflicts.

RESULTS

A summary of data results relative to priority and nonpriority ramp volumes, occupancy levels, and travel time (delay) savings is given in Table 1.

Compliance and violations of priority entry ramps have been historically (5) calculated by the following equations:

$$\text{Compliance (\%)} = [(\text{Total number of nonpriority vehicles using nonpriority ramp}) \div (\text{Total number of nonpriority vehicles})] \times 100\% \quad (1)$$

TABLE 1 Summary of Operational Results (volume, occupancy, delay)

Priority Entry Ramp Site	Avg Nonpriority Volume (vehicles/hr)	Avg Priority Volume (vehicles/hr)		Avg Total Passenger Volume (persons/hr)	Avg Occupancy Nonpriority (persons/vehicle)	Avg Total Combined Occupancy, All Vehicles (persons/vehicle)	Avg Delay Savings (min/vehicle)
		Bus	Van				
Bellaire	875	9	12	1,734	1.20	1.94	4.03
Hillcroft	1,019	15	41	2,495	1.17	2.32	4.92

TABLE 2 Summary of Operational Results (compliance, violations)

Priority Entry Ramp Site	Total Avg Volume Nonpriority Vehicles Using Nonpriority Ramp (vehicles/hr)	Total Avg Volume Nonpriority Vehicles (vehicles/hr)	Compliance Ratio[a] (%)	Total Avg Volume Unauthorized for Priority Entry (vehicles/hr)	Total Avg Volume Vehicles Using Priority Entry Ramp, All Vehicles (vehicles/hr)	Violation Ratio[b] (%)
Bellaire	852	875	97.4	23	42	54.8
Hillcroft	1,010	1,019	99.1[c]	9	65	13.8[c]

[a]Compliance ratio = (Total average volume nonpriority vehicles using nonpriority ramp) ÷ (Total average volume nonpriority vehicles).
[b]Violation ratio = (Total average volume unauthorized for priority entry) ÷ (Total average volume vehicles using priority entry ramp).
[c]Random enforcement at site.

Violation (%) = [(Total number of vehicles unauthorized to use the priority entry ramp)/(Total number of vehicles using the priority entry ramp)] x 100% (2)

A summary of average compliance and violation rate results is given in Table 2. Generally, as the compliance ratio is increased, the violation ratio is decreased. Note the influence of the random enforcement presence at the Hillcroft site, with a 40 percent difference in average violation ratio exhibited between the Bellaire and Hillcroft sites.

The service time of nonpriority vehicles and time headway between arrival of priority vehicles was used to calculate the factor defined as the priority entry exposure ratio. This factor is the time ratio a nonpriority vehicle is exposed to a confirmed use of the priority entry ramp by an HOV. This associated equation is as follows:

Priority entry exposure ratio = {[Service time of nonpriority vehicles (min)]/[Arrival time headway of priority vehicles (min)]} (3)

The hypothesis that follows is that as the exposure ratio increases, the violation rate will decrease. The exposure ratio is influenced by both delay time to nonpriority vehicles and the volume of the priority entry ramp, which confirms the worth of the priority entry ramp and acts as an incentive for modal shift.

A linear-regression model was developed from the data and was tested for significance with the exposure ratio established as the independent variable and the violation rate established as the dependent variable. The correlation coefficient (R^2) was calculated to be 0.6659 (acceptable significance). The linear equation is given as follows:

Violation rate = 0.55 - 0.08 (exposure ratio) (4)

Figure 1 shows a plot of the actual versus predicted data. It should be noted that, even with the calculated significance of these results, the study sample size is limited and no extrapolation may be made outside the limits of the observed data.

SUMMARY AND CONCLUSIONS

The average violation was measured to exceed approximately 40 percent for the designated study sites under investigation. Two factors were assessed as influencing the violation rate. First, random intermittent enforcement appears to decrease the violation rate to approximately 15 percent, as evidenced by the limited comparative data within this study. This level of violations--10 to 15 percent--was reported by previous research (6) as an acceptable standard on priority bypass lanes that use reasonable enforcement.

Second, violations appear to decrease with an increase in the priority entry exposure ratio, which is defined as average service time of nonpriority vehicles divided by average arrival time headways of priority vehicles. A linear relationship was determined (Equation 4) with correlation coefficient (R^2) equal to 0.6659.

As stated, the priority entry exposure ratio is influenced by both delay time to nonpriority vehicles and volume on the priority entry ramp. Measured delay times used in the calculation of the exposure ratio varied from approximately 2.0 to 6.0 min. It is only within this range of delay time that the linear relationship between violation rate and priority entry exposure ratio has been tested.

At some undetermined minimum delay time less than 2.0 min, it is reasonable to expect that violations will decrease because the delay, and the resulting frustration, is not excessive and will be sustained without unauthorized use of the priority ramp. However, it is also difficult, below this minimum delay level, to politically, economically, or operationally justify the priority entry ramp. The incentive for modal shift and priority treatment is questionable. Even if violations exist, the impact is inconsequential; therefore, enforcement may not be required or justified.

At some undetermined maximum tolerable delay time greater than 6.0 min, confirmation of the priority ramp by exposure (volume) will have little impact. Delay time has reached an excessive level, which induces diversion (violations) by nonpriority vehicles. Increased enforcement to control violations is not only required, but the added cost incurred may be offset by the delay savings to priority vehicles.

A graphical representation between the parameters of nonpriority delay and priority vehicle headway relative to enforcement necessary to affect an acceptable violation rate is shown in Figure 2.

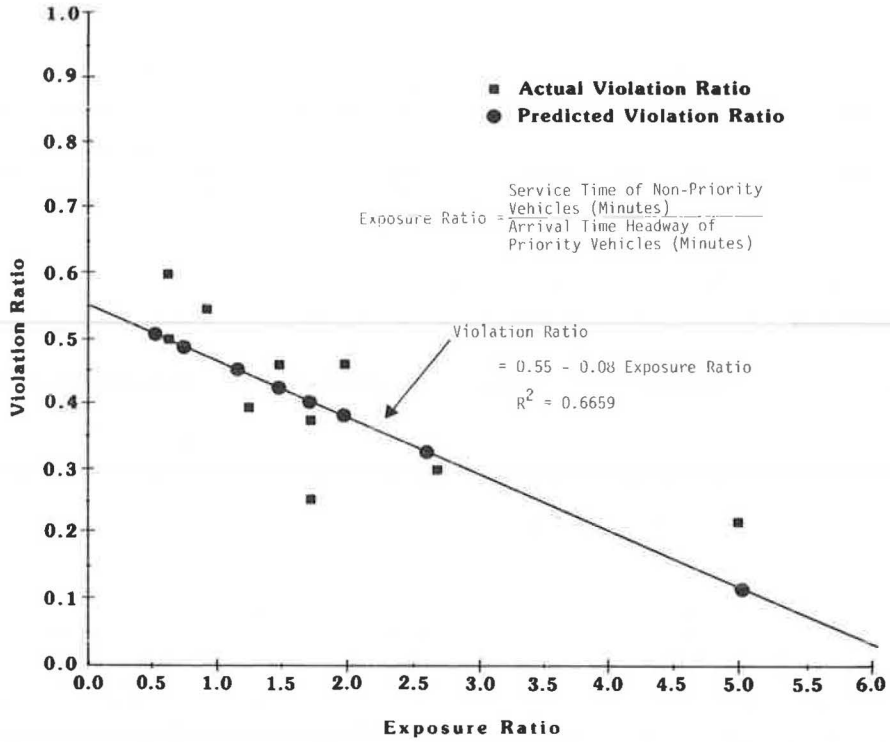

FIGURE 1 Violation ratio versus exposure ratio.

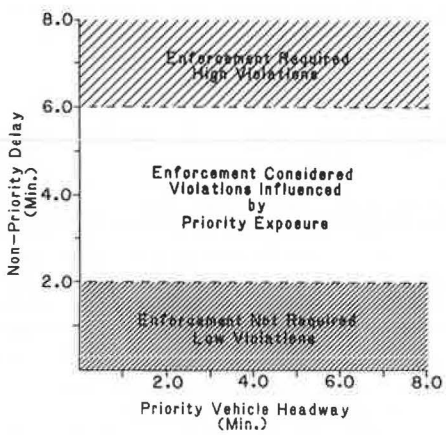

FIGURE 2 Recommended enforcement relative to delay savings and volume on priority entry ramps.

ACKNOWLEDGMENT

This study was sponsored by the Texas State Department of Highways and Public Transportation and was part of an overall project effort entitled "Priority Use of Transportation Facilities." The objective of this research is to provide assistance to the Department in planning and implementing priority treatment techniques for HOVs. The study supervisor is Dennis L. Christiansen, whose direction in this research investigation was greatly appreciated.

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Abridgment

Estimating the Impacts of Ramp-Control Programs

NANCY L. NIHAN and GARY A. DAVIS

ABSTRACT

Time-series designs offer flexible, low-cost tools for evaluating transportation improvements. Some preliminary estimates of ramp-control impacts on an urban freeway are presented.

Traffic engineering studies (1) provide useful tools for evaluating transportation improvements, but because these studies usually require special commitments of resources to gather and process data, they are unsuited for day-to-day monitoring of the transportation system. Recent advances in computer technology have made automatic data collection feasible for freeway surveillance and control (2), transit system monitoring (3), and intersection control (4). With such low-cost data sets, research designs based on time-series analysis (5,6) can be used routinely to evaluate transportation system improvements. A workable methodology for analyzing such data, despite missing values and statistical dependencies, has been described elsewhere (7,8). In this paper some pilot results that describe the effects of on-ramp controls on Seattle's Interstate 5 (I-5) are presented.

STUDY DESIGN

After discussion with employees of the Washington State Traffic Systems Management Center (TSMC), controlled and uncontrolled locations on I-5 were selected. The controlled location was within the section of I-5 subject to ramp controls, whereas the uncontrolled location was about 5 miles south of the controlled location and about 1.5 miles south of the entire ramp-control region. Data from the uncontrolled location were used as a proxy for exogenous effects such as seasonal trend, fuel price changes, and weather. Thus the uncontrolled location provided a covariable in the sense of analysis of covariance. Five-minute volume counts from 6:00 to 10:00 a.m. for July 1, 1981, through December 17, 1981, were provided by the TSMC. The ramp-control program began on September 30, 1981.

Equation 1 gives the linear regression model that was used to estimate the ramp control impacts:

$$DV_t = b_0 + b_1 IN_t + b_2 CO_t + u_t \quad (1)$$

where

DV_t = dependent variable value on day t;
IN_t = 0 on days before 9/30/81, and 1 on days after 9/30/81;
CO_t = covariable value on day t;
u_t = regression residual on day t; and
b_0, b_1, b_2 = regression coefficients to be estimated.

The intervention variable allows the estimation of changes that correspond to the onset of the ramp-control program. For each dependent variable, the regression coefficients were first estimated by using ordinary least-squares and, when necessary, reestimated by using special methods for handling time-series with missing values. Details can be found elsewhere (6-10).

PRELIMINARY RESULTS

A first study investigated the effect of the ramp controls on peak-period traffic volumes. The peak-hour volume and the volume from 6:30 to 8:00 a.m. (the time period during which the ramp controls operated after September 30, 1981) were calculated for each day for both the controlled location and the study location. These results are given in Table 1. Surprisingly, the volumes decreased in response to the ramp controls. This effect could result from the ramp controls acting to keep volumes at somewhat below capacity, thus improving travel times by restricting access to the freeway. A more detailed investigation of this point is in progress.

TABLE 1 Effects of the Ramp-Control Program

Dependent Variable	Control Effect (vehicles)	Control Effect (vehicles/lane/hr)	Significance
Peak-hour volume	-384.3	-96.1	$p < 0.01$
6:30 - 8:00 a.m. volume	-582.4	-97.1	$p < 0.01$
6:00 - 6:30 a.m. volume	96.8	48.4	$p < 0.01$
8:00 - 8:30 a.m. volume	9.3	4.7	$p > 0.05$
6:00 - 10:00 a.m. volume	-276.6	-17.3	$p > 0.05$

It was also of interest to know whether this volume decrease resulted in temporal or spatial demand shifts; that is, are travelers using a different route or the same route at a different time? Traffic volumes from 6:00 to 6:30 a.m., 8:00 to 8:30 a.m., and 6:00 to 10:00 a.m. were calculated, and ramp-control effects were estimated by using Equation 1. These results also appear in Table 1. The 6:00 to 6:30 a.m. period has a definite increase in volume, whereas the other two periods have no change. It appears then that the ramp controls have flattened the morning peak by shifting some trips to off-peak times.

CONCLUSION

Fine-tuning a ramp-control program to keep it current with the evolution of an urban system will usually require more precise information than that available from simple before-and-after studies. In this paper it has been demonstrated how time-series methods can provide some of this information at the required level of precision. These topics are currently being investigated in greater detail.

ACKNOWLEDGMENT

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Description of a Combined Approach for Arterial Signal Coordination Considering Bandwidth and Delays

K.G. BAASS and B. ALLARD

ABSTRACT

The coordination of traffic lights on arterial streets can be achieved by maximizing the bandwidth or by trying to minimize delays and stops encountered on the artery and on the side streets. Both of these approaches have advantages that are partly retained in a compromise solution, where a green band is selected from among several possible bands so as to cause the least delay to the driver. A first step in this direction is described by giving the outline of a program that analyzes the relation among delays, speeds, offsets, bandwidths, and cycle lengths over a wide range of speeds and cycles. This program can be used by the practitioner to obtain the speed (for a given cycle length) that maximizes the bandwidth, or to determine the cycle (for a given speed) that maximizes bandwidth. It can also be useful to find the offset, speed, or cycle length that causes a reasonable delay to the driver while retaining an acceptable green band. The program that combines bandwidth maximization and delay minimization was applied to five data sets taken from the literature. It was found that the new approach is feasible and could have economic advantages. It was further found that, for low traffic volumes and no platoon dispersion, there is a relation between bandwidth and delays; larger bands generally result in less delay than smaller ones. But as dispersion and traffic volumes increase, this relation no longer holds. It was also strongly confirmed in all cases studied that delays increase with longer cycle lengths for a given value of $K = C \cdot V$, and it was shown that there is a tendency for increasing delay with increasing cycle length for a given speed on the artery.

Traffic signal coordination on arterial streets that are not part of a network remains an important problem for the traffic engineer. Two approaches to the coordination of traffic lights are available: bandwidth maximization, and the minimization of a disutility function, which is measured in terms of delays, stops, fuel consumption, and air pollution. Both of these ways of solving the problem have advantages, and an approach that combines these two methods would be of interest. This would retain the undoubtedly important advantage of the psychological effect of the green band and, at the same time, would be more efficient in terms of delays and stops and reducing fuel consumption and time lost by the driver.

There are many variables that intervene in the solution of the problem. Delays and stops on a coordinated artery depend on signal settings, offsets and bandwidth, platoon size, dispersion, and platoon speed. Relatively little is known about the relationships between these variables. Certain intuitive notions, such as the idea that the larger

the bandwidth the lower the delay encountered on the artery, are commonly perceived by the user but have not been conclusively demonstrated.

The aim of the present paper is to describe the development of an algorithm and a computer program based on an approach that combines bandwidth maximization and disutility minimization. Some of the results obtained by applying this program to five arterial data sets are given. Because the number of relations between the intervening variables is high, only the most important relations were studied. The results indicate that certain economic benefits for drivers can be achieved by applying the proposed procedure.

COMBINED PROGRAM

The program developed contains two parts. The first part determines the bandwidths and offsets over a wide range of speeds and cycles, whereas the second part evaluates the disutility function for the bands and offsets developed in the first part. This is done by simulating traffic flow through the artery. In order to allow the investigation of as many relations between the intervening variables as possible, it was decided not to use a microscopic simulation model. The more macroscopic simulation approach used by Robertson (1) is still detailed enough to represent an average platoon, and this was incorporated into the evaluation part of the program. It has been shown in many applications that this provides satisfactory results. The initial development of these two subroutines was carried out by Couture (2), who also conducted the first tests.

The objective of the combined approach is to choose an offset that provides a green band of satisfactory width (not necessarily the maximum width for a given speed) and that ensures, at the same time, less delay and fewer stops than other bands of comparable width.

Bandwidth Algorithm

The bandwidth algorithm is based on the procedure described by Baass (3). Certain basic relations exist that depend on the geometry of the problem and that are useful for the development of an efficient algorithm. It was proved by Little et al. (4) that for equal speeds and volumes in both directions, a maximal bandwidth is given when the offsets between lights are 0 or half a cycle (semi-integer coordination). Bands with unequal speeds and volumes can be derived from this optimal solution by geometric transformations. In a first phase the bandwidth-generating algorithm was thus limited to a constant speed on the artery in both directions and a constant flow of traffic on the artery. These conditions can be changed by using the adjustments described by Little et al. (4).

The geometry of Figure 1 shows that, for all combinations of $V \cdot C = K$, the bands have to have the same widths. K can be interpreted as a scale constant, and it is the distance traveled at a speed v (m/sec) during the cycle time C (sec).

In order to obtain a complete understanding of the relationships between the variables, the program allows for a wide range of K (where $K = V \cdot C$ is a scale constant) for which bands and offsets are determined. Describing the bands and offsets from K = 800 to K = 12,000 covers a range of speeds between V = 20 and 100 km/h and cycles from 40 to 120 sec.

The algorithm is based on the interference approach described by Brooks (5), although this had to

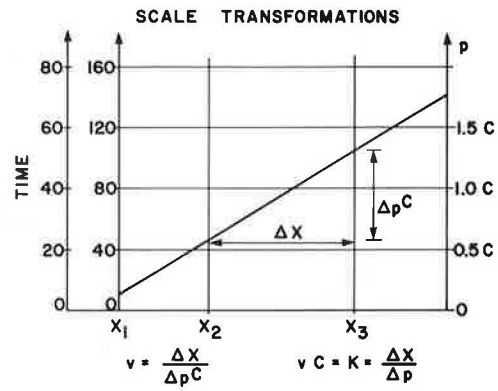

FIGURE 1 Scale transformations.

be modified to take into account two problems not considered by Brooks or Little et al. The first problem concerns the occurrence of multiple bands that are encountered frequently. The presence of multiple bands may have an important impact on delays and stops and may be altogether undesirable from the standpoint of bandwidth efficiency. The second problem is frequently encountered in the case of arteries with nearly equal distances between intersections and with the same red times at each light (as may happen in regular grids). In this case several bands of equal width (for a given speed) are obtained by different offset schemes, which generate different delays and stops. The incorporation of these two elements into the bandwidth algorithm is briefly outlined and illustrated by two simple examples. The detailed algorithm is described elsewhere.

Little et al. (4) have shown that in the case of semiinteger coordination there are 2**(n - 1) possible offset schemes to be considered for determination of a maximum band for a given speed and cycle. These offset schemes can be described, for simplicity of representation and for rapidity of calculations, by using the binary system. Consider n intersections; the offset of the first intersection is arbitrarily set to 0 and the intersections are numbered from 1 to n from left to right. If a light is in phase with the first light, its offset is 0; if not it is 1. The following offset scheme represents a sequence of binary numbers, and the decimal equivalent describes the offset scheme in a unique way:

0 1 0 1 1 1 1 0 1 0

This offset scheme is represented by the number 378. Offset scheme 9 would indicate that light n and light (n - 3) are out of phase. This simplified representation is essential for the rapid execution of the program.

The principles of the modified algorithm can best be illustrated by considering Figure 2.

The initial calculations are the same as those of Brooks as described by Gazis (6). The light with the minimum green is determined from which the maximum possible band can be identified. Its slope is defined by the speed. The red lights will interfere at the top (left) and at the bottom (right) with this maximal band, because the red lights can have two positions on the 50 percent cycle horizontals in the time-space diagram. The left and right interferences are easily calculated together with their corresponding offsets with respect to the first intersection. The left interferences can then be ordered by decreasing magnitude. The data in Table 1 describe the situation shown in Figure 2.

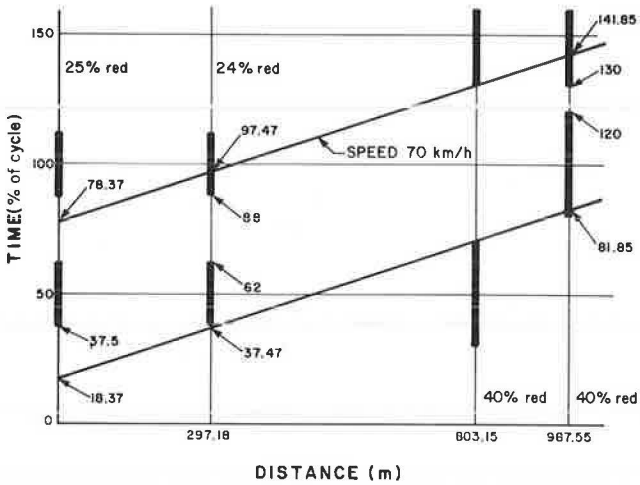

FIGURE 2 Determination of interferences; artery in Laval, Quebec; data set 2.

TABLE 1 Interferences and Offsets for Example in Figure 2

I	LIGHT (K)	LEFT(I)	RIGHT(I)	THETA(K) LEFT	RED(K) %
1	1	40.87	0.00	0	25
2	4	11.85	38.15	0	40
3	2	9.47	24.53	1	24
4	3	0.00	0.00	0	40

The enumeration of all possible bands could be done in a systematic way based on these interferences, but it is evident that this is an infeasible approach. An efficient way must be found to generate only the largest bands because, for delay evaluation purposes, there is interest in the band of maximum width and in all bands B that are wider than an acceptable minimum. This requirement can be stated as an inequality: $X \cdot Bmax \leq B \leq Bmax$, where X is a value between 0 and 1 chosen by the user. The procedure adopted for calculating these bands and the corresponding offsets is a good heuristic. It calculates 2(n - 1) bands, which ensures that at least the n largest bands for each speed are found, together with the corresponding offsets. The efficiency of the heuristic was verified in all of the examples tested in the five data sets.

The first band is calculated considering only the offsets, thus producing the left interferences and a bandwidth of BAND = MINGREEN - LEFT(1). The next (n - 1) bands are obtained by exchanging right interferences with left interferences (on the dotted diagonal in Table 1) by always using the maximal right interference for all j < i. The next (n - 2) bands are obtained by exchanging right interferences with left interferences on the solid diagonal (Table 1). Multiple bands are possibly present when the interferences are larger than the corresponding red times. The double band is given by the difference between the interference and the red time minus the largest interference of the remaining lights that have their interferences on the same side. This is easily verified in Figure 2. The data in Table 2 give the calculations required to obtain the n largest and double bands.

The algorithm always produces the maximum band and in most cases the n largest bands. This holds especially for n bands of equal width. This can be seen in Figure 3 for the special case of equal distances between intersections. The data in Table 3

TABLE 2 Calculation of the N Largest Bands

CALCULATIONS	BAND	OFFSET	NUMBER
60-40.87 40.87-25-11.85	19.13 4.02	0 1 0 0	4
60-0.0-11.85	48.15	0 0 1 1	3
60-38.15-9.47	12.37	0 0 1 0	2
60-38.15-0.0	21.85	0 1 1 0	6
60-40.87-38.15 40.87-25-9.47	0.0 6.40	0 1 0 1	5
60-11.85-24.53 24.53-24-0.0	23.62 0.53	0 1 1 1	7

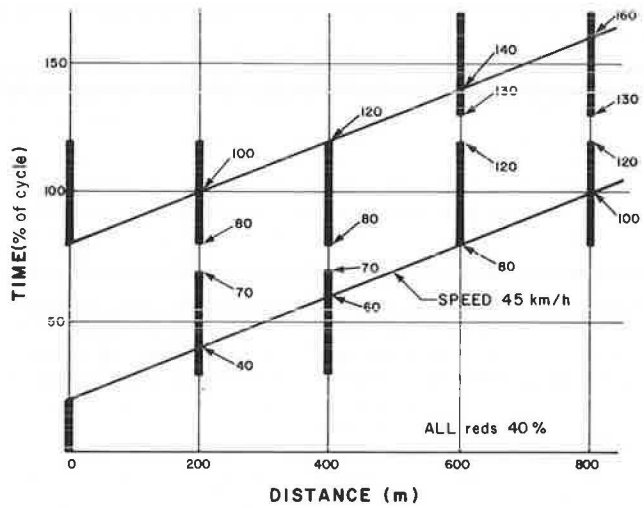

FIGURE 3 Determination of interferences for an artery with equal distances between intersections.

TABLE 3 Calculation of Bands and Offsets

I	LIGHT (K)	LEFT(I)	RIGHT(I)	THETA(K) LEFT	RED(K) %
1	3	40	10	0	40
2	5	30	20	1	40
3	2	20	30	0	40
4	4	10	40	1	40
5	1	0	0	0	40

BAND %	OFFSET	N	DELAY 850v/h	DELAY 1700v/h
20	0 0 0 1 1	3	14.9	69.4
20	0 0 1 1 1	7	17.2	72.4
20	0 0 1 1 0	6	16.7	67.1
20	0 1 1 1 0	14	17.2	72.4
20	0 1 1 0 0	12	14.9	69.4
0	0 0 0 1 0	2	24.4	77.7
0	0 1 1 1 1	15	28.1	74.6
0	0 0 1 0 0	4	24.4	77.7

give the necessary calculations. There are five maximal bands with quite different delays.

Evaluation of Delays and Stops

The evaluation of traffic performance is done by determining delays, stops, and an index of performance that is defined in the same way as in TRANSYT (1). The traffic is input at the beginning of the artery in a uniform pattern, and the platoon is followed downstream while considering entering and exiting vehicles at the intersections (uniform entries over green period from the side streets). The platoon is dispersed ($F = 1/1 + kt$) by using Robertson's formula (1). The procedures for calculating delays and stops are similar to those used in TRANSYT, in which interval size can be varied. The delays encountered on the side streets are determined by Webster's formula (7), and the stops on the side streets are estimated as in the SOAP (8) program. The timings and offsets of lights as well as platoon speeds are transferred from the first part of the program to this evaluation procedure.

The program produces graphic and corresponding printed output (see Figure 4). There are four different graphics illustrating the relations among

1. Band, speed, and delay for a given cycle length; the delay is the minimum delay calculated for all bands $X \cdot Bmax \leq B \leq Bmax$;
2. Delay and cycle for different values of $K = C \cdot V$;
3. Delay and cycle for different speeds; and
4. Bandwidth and cycle for different speeds.

These outputs can be used in practical applications or for research purposes. The execution times of the program are reasonable, considering the repeated delay evaluations for different offsets and for each speed (interval for speed evaluation 0.2 km/h for $10 < V < 125$ km/h, interval for cycle evaluation 5 sec for $40 < C < 120$ sec, interval for delay evaluation 1 sec). The example artery cited by Little (4) of 10 intersections necessitates 1.5 min of execution time on an IBM 4341 computer.

ANALYSIS OF RESULTS

To clarify some of the relationships between the different intervening variables an experiment was conducted by using the program and fixing the values of certain variables and letting only one variable change at a time. Delay, a basic indicator of performance, was studied in relation to speed, bandwidth, cycle length, and volume. This was done for five data sets, which are

1. An artery with 5 intersections at equal distances (200 m) and 40 percent of red time at each intersection;
2. An artery in Laval, Quebec, cited by Baass (3), with 4 intersections;
3. The artery used in MAGTOP (9) with 5 intersections;
4. The artery described by D.A. Bowers in the ITE manual (10, Figure 96, pp. 234), with 8 intersections; and
5. The artery that served as an example for Little (4), with 10 intersections.

There are certain traffic conditions that could be termed ideal for the application of the bandwidth approach. These correspond to its basic hypotheses. The conditions would include few vehicles on the artery, low degree of saturation, no platoon dispersion, constant platoon speed, and few vehicles entering the artery from the side streets. Given these conditions, it could be intuitively hypothesized that a larger band would cause fewer delays and stops than a smaller one. This was the first point to be investigated. The second was to verify, on a larger sample of arteries and over a wider range of cycles, the hypothesis formulated by Kahng and May (11), Mao et al. (12), and Rogness (13), which states that delay would increase on an artery when cycle length increases. This hypothesis was studied with respect to bandwidth, offsets, and to the constant K. The third point was to investigate the relation among a given speed, the cycle length, and delay; it was hypothesized that, for a given speed, a shorter cycle length is preferable with respect to delay.

Because these hypotheses are most likely to be true on a near ideal sample artery, the cases given in Table 4 were studied on the first data set.

Figure 5 shows the relationship between speed, bandwidths, and delays for an 80-sec cycle or for $K = 800$ to $K = 10{,}000$ for a volume of $Q = 212$ vehicles per hour. There is less delay as compared with neighboring values for each peak in the bandwidth curve. But clearly, comparable bands do not necessarily have comparable delays. Delays also depend on the speeds and offsets. In fact, the offset element is an important one. Each of the regions around the peaks in this graph have particular offset schemes [see Baass (3)]. They are given, for this example, in Table 5.

It can be seen that the offsets remain the same over a wide range of speeds, and that this has an influence on the delays. In this particular case there is a peak in bandwidth at the speed of 18 km/h, where the bandwidth is equal to the minimum green; the only delay encountered for the platoon is at the first light in each direction of in-bound and out-bound bands because there is no dispersion. Generally speaking there is a relation between bandwidth and delay, which is also illustrated in Figure 6. For a given value of K, wider bands do produce less delay at this level of volume. Furthermore, it is clearly seen that there is a relationship between cycle and delay (which should not come as a surprise, considering the way delays are calculated). Because K is given by the product of the cycle length and the speed (in Figure 5), and the delay decreases (in Figure 6) for a given value of K, these figures can be used to choose the best values of speeds and cycles. A practical illustration is given later in this paper. Figure 7 shows a general trend of increasing delay for a given speed as cycle lengths increase. The peak of the bandwidth in Figure 8 is reflected as a low delay in Figure 7 and, as an exception, the cycle of 80 sec gives the lowest delay for a speed of 18 km/h. From this graph it is possible to find the cycle that minimizes the delay for a given speed and that retains a band whose width is shown in Figure 8. This figure also gives the cycle length that maximizes the bandwidth for a given speed.

As volume increases to 850 vehicles per hour, the curves still behave in much the same way as at $Q = 212$ vehicles per hour. However, the bandwidth factor becomes less important as differences in delays disappear between curves of equal K. A further increase of volume to 1,700 vehicles per hour (still having no platoon dispersion) is illustrated in Figures 9-11. Bandwidths have no further impact on delays, except for the speed of 18 km/h, where the band corresponds to the minimum green. It would be desirable to achieve bands that correspond to the width of the minimum green, which is not necessarily

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                    I M P R E S S I O N   D E S   D O N N E E S

NOMBRE DE CARREFOURS:                          8
DUREE DU CYCLE MIN.,MAXI.,INT (SEC)           40   130    5
NOMBRE DE VITESSES UTILISEES                   5
VALEUR DES VITESSES:    17.7   27.5   35.8   53.5   71.1   0.0   0.0   0.0   0.0   0.0   0.0   0.0   0.0   0.0   0.0
NOMBRE DE CONSTANTES UTILISEES                 5
VALEUR DES CONSTANTES:  1416   1880   2848   4280   5704    0     0     0     0     0     0     0     0     0     0
INCREMENT POUR LA DISPERSION DES PELOTONS:   1.00
FACTEUR MULTIPLICATEUR DES ARRETS POUR
LE CALCUL DE L'INDICE DE PERFORMANCE:          8
FACTEUR DE PONDERATION POUR LES VOLUMES:     1.00
NUMERO DE L'ENSEMBLE DE DONNEES:            BOWERS


CARREFOUR                          V O L U M E S

          ---------- ARTERE ----------   ---------- RUE SECONDAIRE ----------
     MONTANT VIR.GAU VIR.DRO DESCEND VIR.GAU VIR.DRO MONTANT VIR.GAU VIR.DRO DESCEND VIR.GAU VIR.DRO

  1    850.    0.      0.    850.     0.      0.      0.      0.      0.      0.      0.      0.
  2    850.    0.      0.    850.     0.      0.      0.      0.      0.      0.      0.      0.
  3    850.    0.      0.    850.     0.      0.      0.      0.      0.      0.      0.      0.
  4    850.    0.      0.    850.     0.      0.      0.      0.      0.      0.      0.      0.
  5    850.    0.      0.    850.     0.      0.      0.      0.      0.      0.      0.      0.
  6    850.    0.      0.    850.     0.      0.      0.      0.      0.      0.      0.      0.
  7    850.    0.      0.    850.     0.      0.      0.      0.      0.      0.      0.      0.
  8    850.    0.      0.    850.     0.      0.      0.      0.      0.      0.      0.      0.


          DEBIT DE SATURATION
     --- ARTERE ---   -- RUE SEC. --
     MONT.    DESC.    MONT.    DESC.

  1   3400.    3400.    3400.    3400.
  2   3400.    3400.    3400.    3400.
  3   3400.    3400.    3400.    3400.
  4   3400.    3400.    3400.    3400.
  5   3400.    3400.    3400.    3400.
  6   3400.    3400.    3400.    3400.
  7   3400.    3400.    3400.    3400.
  8   3400.    3400.    3400.    3400.


LIENS    VALEUR DE LA CONST. DE ROBERTSON

  1   0.3500   0.3500
  2   0.3500   0.3500
  3   0.3500   0.3500
  4   0.3500   0.3500
  5   0.3500   0.3500
  6   0.3500   0.3500
  7   0.3500   0.3500


                    ***********************************************************
                    * CARREFOUR *          ABSCISSE          *     ROUGE     *
                    *         *                             *               *
                    ***********************************************************
                    *    1    *            0.0  METRES      * 50.00 POURCENT *
                    *    2    *          213.36 METRES      * 50.00 POURCENT *
                    *    3    *          396.24 METRES      * 50.00 POURCENT *
                    *    4    *          609.60 METRES      * 50.00 POURCENT *
                    *    5    *          676.56 METRES      * 50.00 POURCENT *
                    *    6    *          783.34 METRES      * 50.00 POURCENT *
                    *    7    *          991.46 METRES      * 50.00 POURCENT *
                    *    8    *         1188.72 METRES      * 50.00 POURCENT *
                    *         *                             *               *
                    ***********************************************************
          VERT MINIMUM:  50.00


          DUREE DU CYCLE:    40
     CONST(K):1416.00188 0.002843.004280.005704.00
     VITESSE:    35.42   47.03   71.20  107.00  142.60
     BANDE:      27.12   16.77   33.83   16.66   13.45
     RETARD:     21.61   20.71   19.84   19.78   18.06

          DUREE DU CYCLE:    45
     CONST(K):1416.00188 0.002843.004280.005704.00
     VITESSE:    31.47   41.78   63.29   95.11  126.76
     BANDE:      27.12   16.73   33.83   16.66   13.45
     RETARD:     23.01   22.07   19.42   19.11   21.25

          DUREE DU CYCLE:    50
     CONST(K):1416.00188 0.002843.004280.005704.00
     VITESSE:    28.32   37.60   56.96   85.60  114.08
     BANDE:      27.12   16.73   33.83   16.66   13.45
     RETARD:     26.66   25.19   22.01   22.42   23.85
                         BOWERS DU ITE  8 CARREFOURS  F=0.35   Q=850


                    VITESSE    BANDE    RETARD
VITESSE M.MAX M.MIN    **DECALAGE BANDE RETARD **DECALAGE BANDE RETARD **DECALAGE BANDE RETARD

31.50 10.66 39.60          100     10.7    39.6
31.60 11.10 38.52          100     11.1    38.5
31.70 11.54 38.52          100     11.5    38.5
31.80 11.98 38.52          100     12.0    38.5
31.90 12.41 38.52          100     12.4    38.5
32.00 12.84 38.17          100     12.8    38.2
32.10 13.27 38.17          100     13.3    38.2
32.20 13.69 38.17          100     13.7    38.2
32.30 14.11 38.01          100     14.1    38.0
32.40 14.53 37.86          100     14.5    37.9
32.50 14.95 37.86          100     15.0    37.9
32.60 15.36 37.86          100     15.4    37.9
32.70 15.78 37.39          100     15.8    37.4
32.80 16.19 37.39          100     16.2    37.4
32.90 16.59 37.39          100     16.6    37.4
33.00 17.00 37.39          100     17.0    37.4
33.10 17.40 37.39          100     17.4    37.4
```

FIGURE 4 Partial computer output for data set 4.

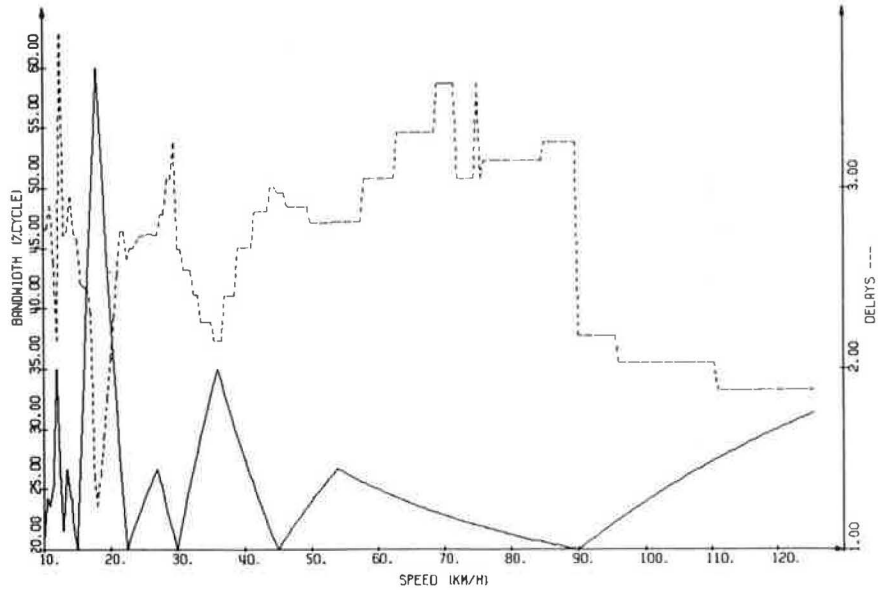

FIGURE 5 Band-speed-delay curves for data set 1 (C = 80 sec, Q = 212 vehicles/hr, k = 0.0).

TABLE 4 Traffic Conditions Studied for the Five Data Sets

VOLUME	SAT.FLOW	DEGREE X	DISPERS. k
212	3400	0.1	0.0, 0.35
450	3400	0.22	0.0, 0.35
850	3400	0.42	0.0, 0.35
1700	3400	0.83	0.0, 0.35

TABLE 5 Offset Schemes Corresponding to Figure 4

SPEED		OFFSET
BEGIN	END	SCHEME
15	22.5	11
22.5	30	10
30	41.1	7
41.1	45	13
45	71.6	8
71.6	90	16
90	125	1

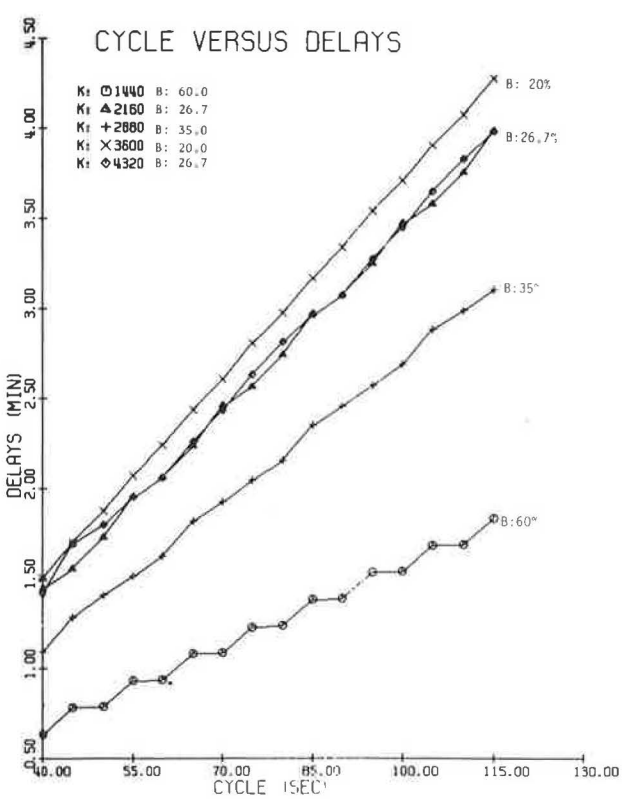

FIGURE 6 Data set 1 (Q = 212 vehicles/hr, k = 0.0).

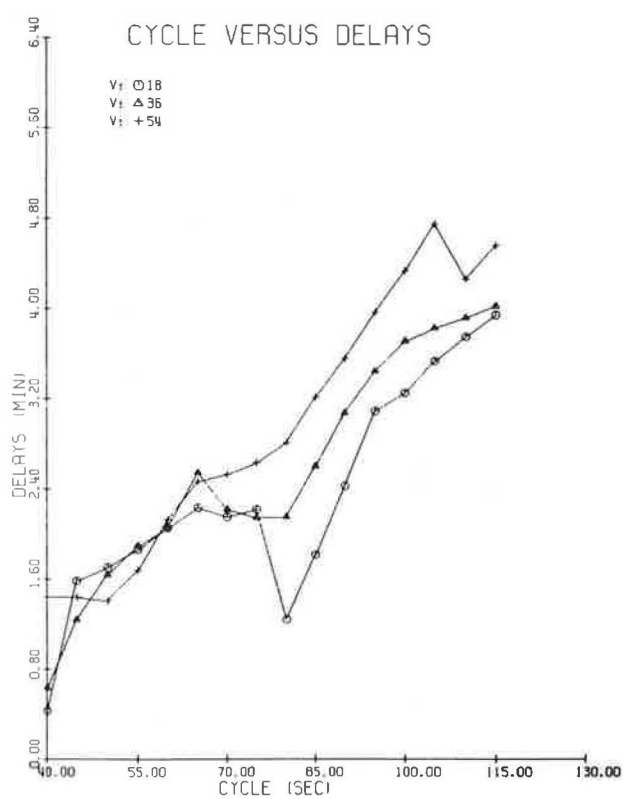

FIGURE 7 Data set 1 (Q = 212 vehicles/hr, k = 0.0).

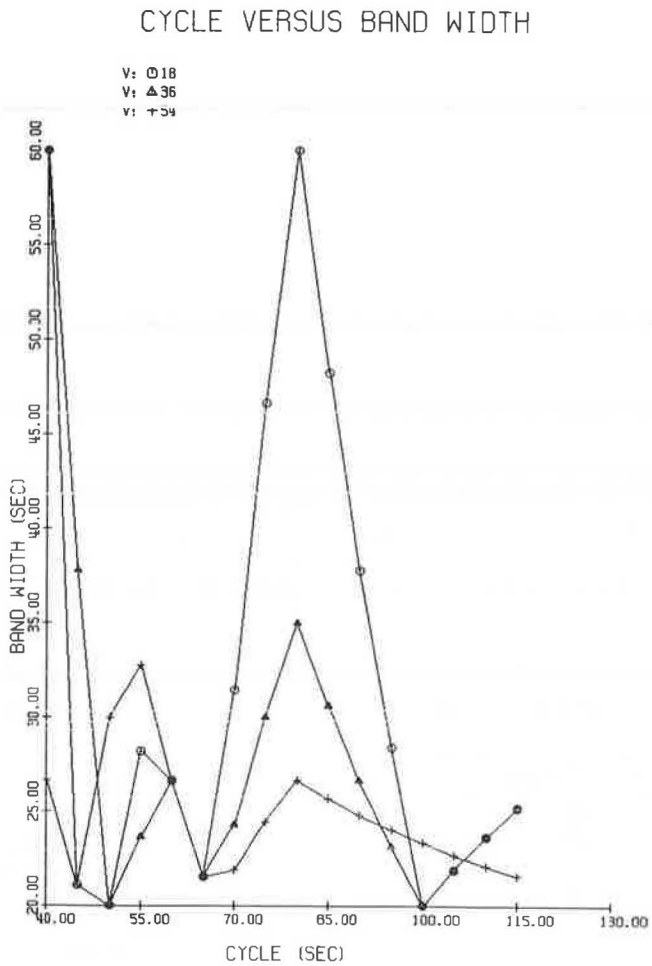

FIGURE 8 Data set 1 (Q = 212 vehicles/hr, k = 0.0).

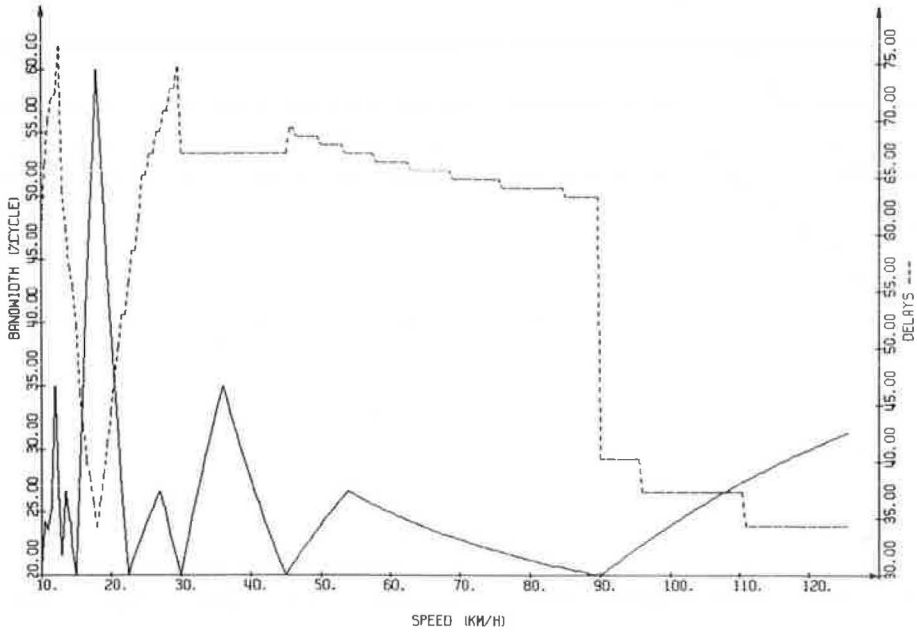

FIGURE 9 Band-speed-delay curves for data set 1 (C = 80 sec, Q = 1,700 vehicles/hr, k = 0.0).

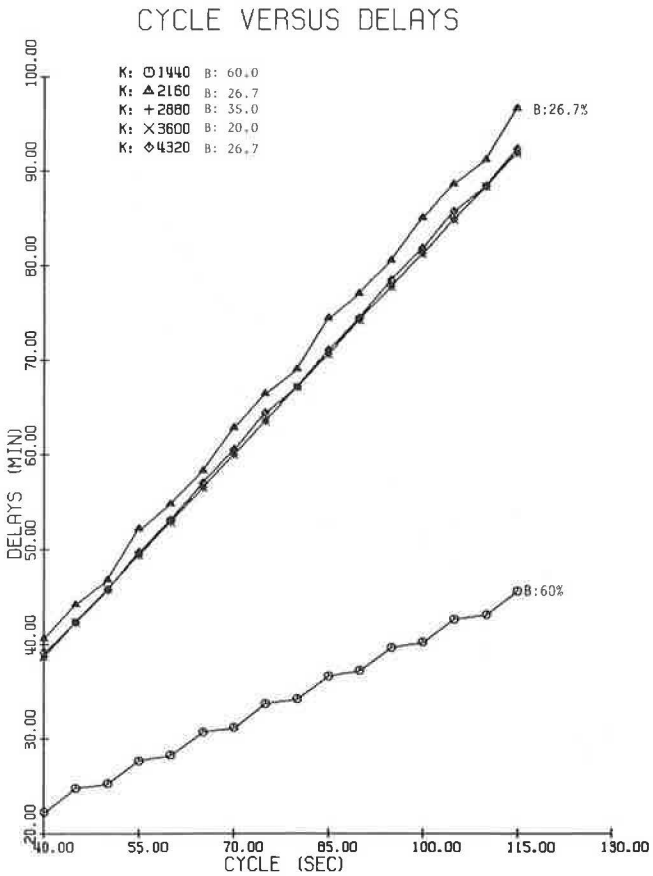

FIGURE 10 Data set 1 (Q = 1,700 vehicles/hr, k = 0.0).

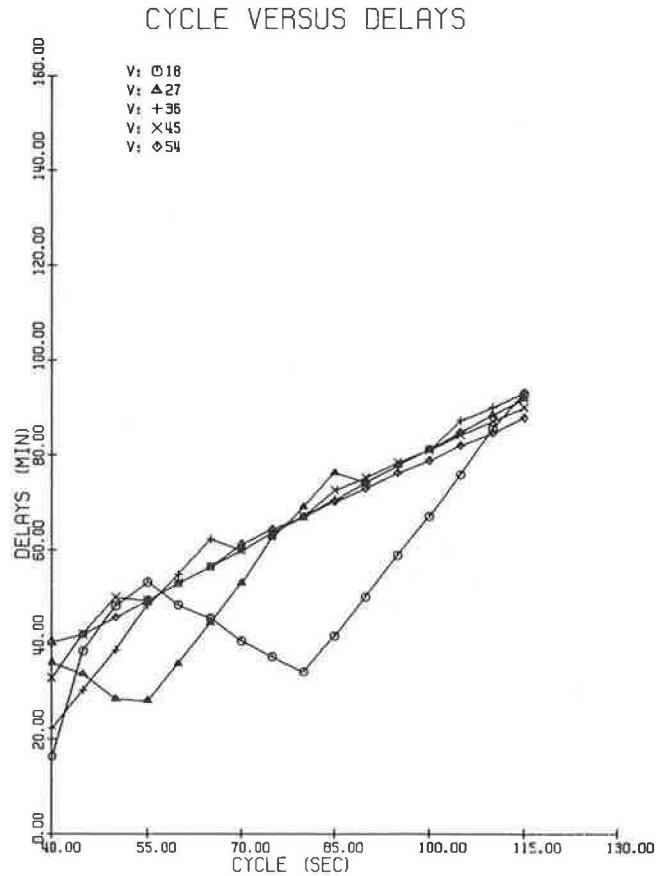

FIGURE 11 Data set 1 (Q = 1,700 vehicles/hr, k = 0.0).

possible, because there are many cases where the geometry of the artery and the red times prevent such a solution.

At low volumes with dispersion (Q = 212 vehicles per hour, k = 0.35), the relationship between bandwidth and delays is still discernible (see Figures 12-14), but as volumes increase this relation no longer holds.

The conclusion is that platoon dispersion alone is sufficient to invalidate the first hypothesis for a general case. However, the relation among cycle length, delay, and K is strongly confirmed. The tendency for delays to increase with increasing cycle lengths for a given speed is also verified in most cases.

The sample data set 2, which represents a more

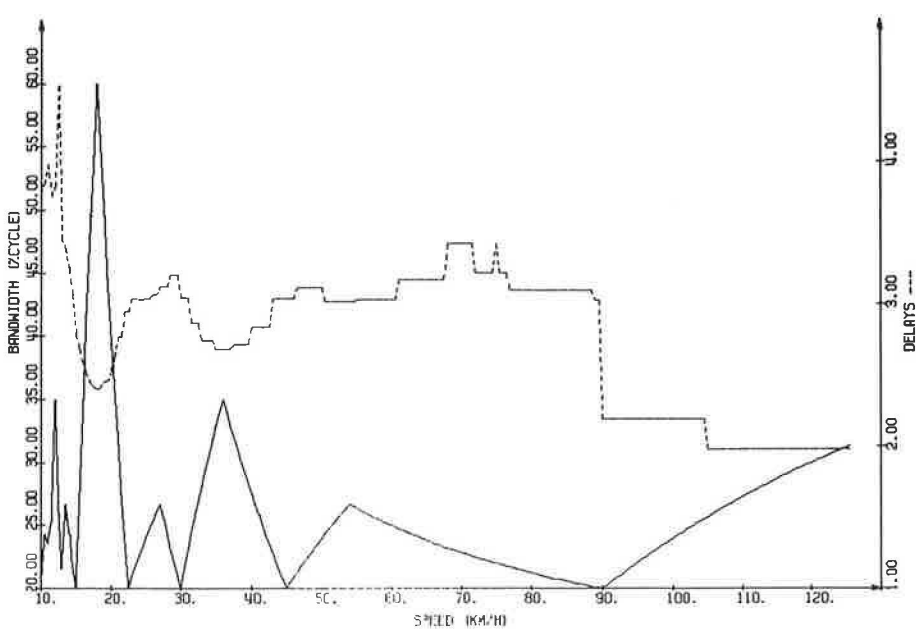

FIGURE 12 Band-speed-delay curves for data set 1 (C = 80 sec, Q = 212 vehicles/hr, k = 0.35).

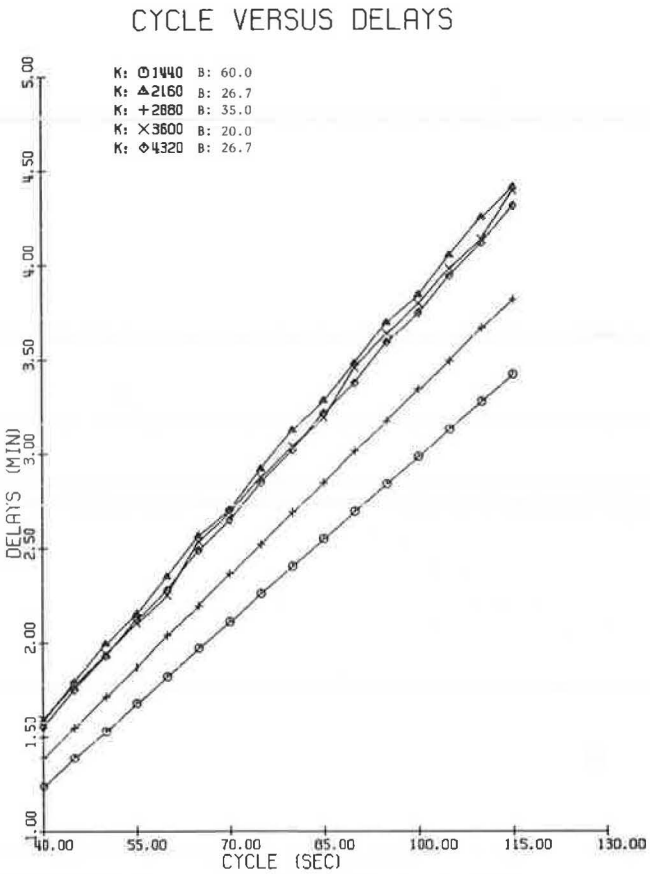

FIGURE 13 Data set 1 (Q = 212 vehicles/hr, k = 0.35).

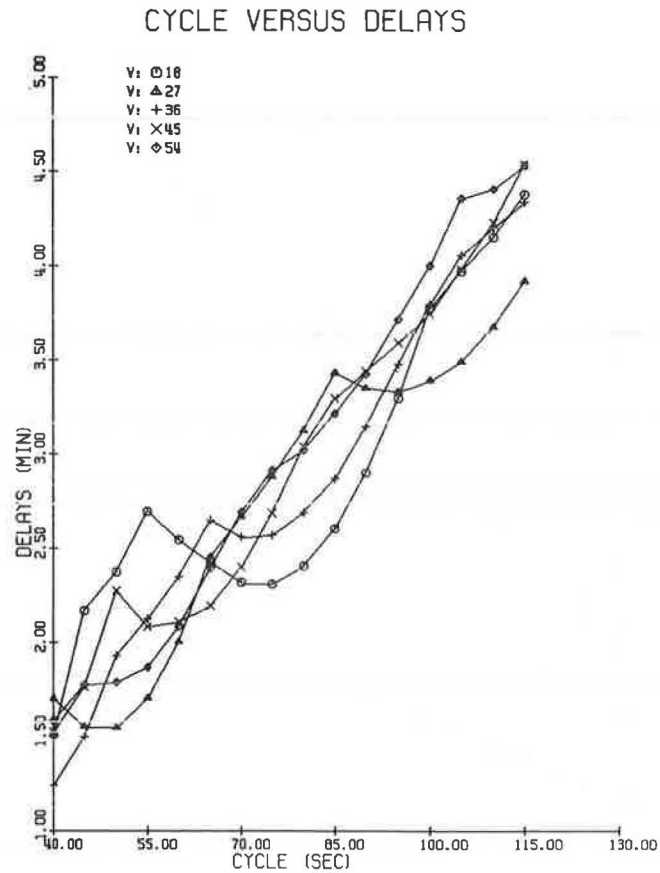

FIGURE 14 Data set 1 (Q = 212 vehicles/hr, k = 0.35).

general artery, was analyzed in the same detail. Only two results are reported here.

At low volumes (Q = 212 vehicles per hour, k = 0.0) and no dispersion, Figures 15-17 show that bandwidths have an influence on delays. The same cannot be said when dispersion is introduced at the same volume. Figures 18-20 for a volume of 850 vehicles per hour and dispersion indicate that larger bandwidths alone do not guarantee lower delays in all cases. But there are regions in Figure 18 that are more interesting than others with respect to bandwidth and delays. Choosing a certain point on the curve and implementing the corresponding offsets alone, however, does not guarantee minimum delays.

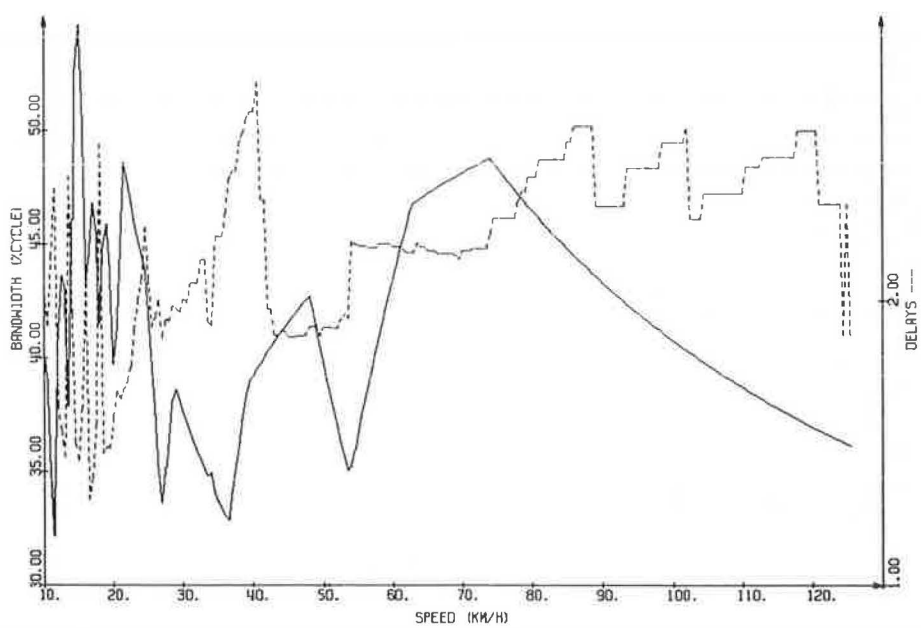

FIGURE 15 Band-speed-delay curves for data set 2 (C = 80 sec, Q = 212 vehicles/hr, k = 0.0).

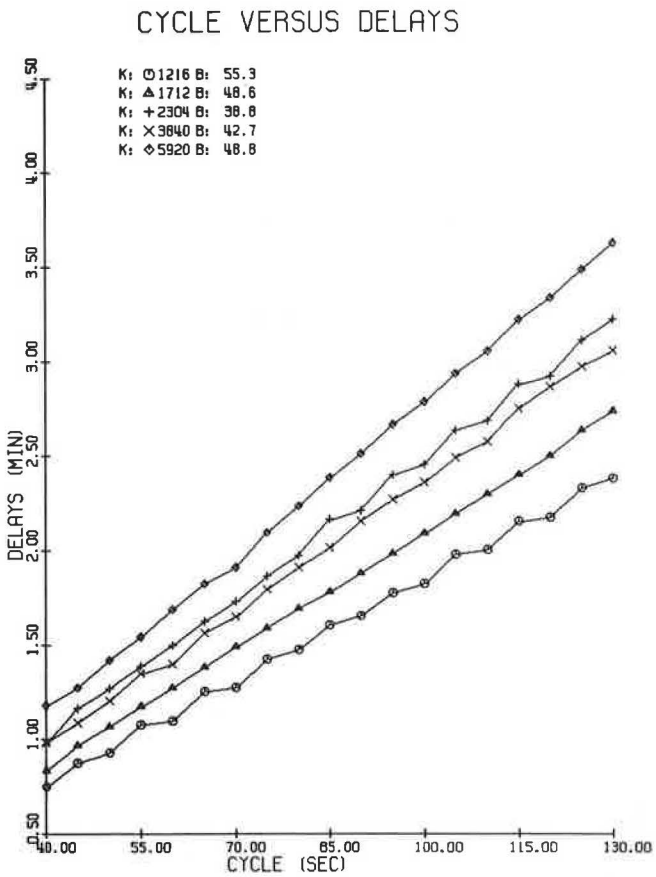

FIGURE 16 Data set 2 (Q = 212 vehicles/hr, k = 0.0).

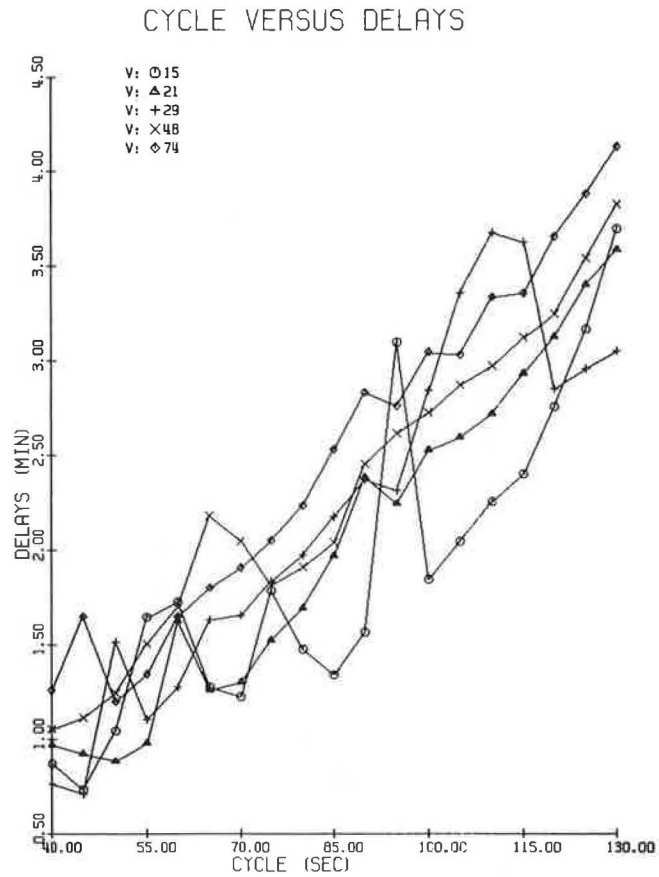

FIGURE 17 Data set 2 (Q = 212 vehicles/hr, k = 0.0).

Because certain portions of the curve have the same offset (see Table 6 for an example), different platoon speeds are possible, which have bands from 32.8 to 42.7 percent in width. But the corresponding delays may vary widely. It would be necessary to indicate to the driver, through traffic signs, the speed that minimizes delay.

Two further examples illustrate the points made. These are both arteries with 850 vehicles per hour of traffic volume and a degree of saturation of 0.40 with a dispersion of k = 0.35. The example cited in MAGTOP is illustrated in Figures 21-23, and the artery described by Little et al. (4) is illustrated in Figures 24-26. The latter example shows that the

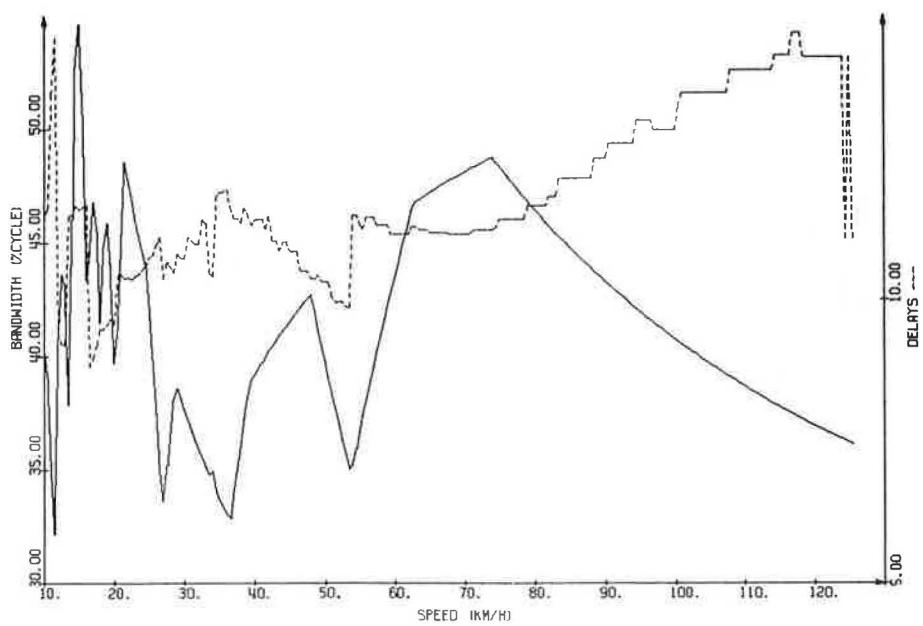

FIGURE 18 Band-speed-delay curves for data set 2 (C = 80 sec, Q = 850 vehicles/hr, k = 0.35).

TABLE 6 Domain of Offset Scheme 5

OFFSET SCHEME 5		
SPEED	BAND	DELAY
36.40	32.80	11.76
48	42.7	9.94
53.5	35.1	9.55

bandwidth is not necessarily in phase with the delays (large band, low delay), but in all cases there is a strong relationship between cycle lengths and delays.

The program can also be useful in practice to help the engineer to choose offsets, speeds, and cycle lengths. In practical applications there are three possibilities with respect to the choice of the cycle and the speeds on an artery.

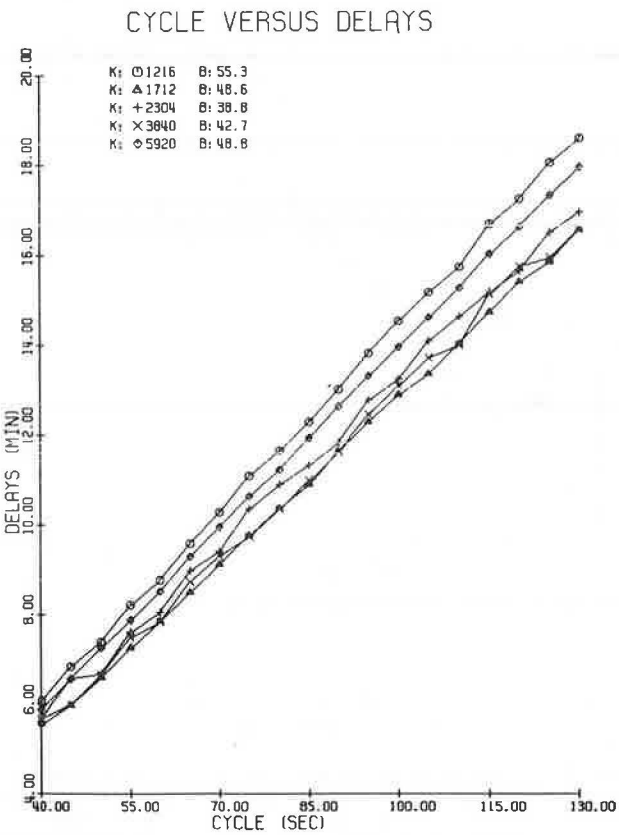

FIGURE 19 Data set 2 (Q = 850 vehicles/hr, k = 0.35).

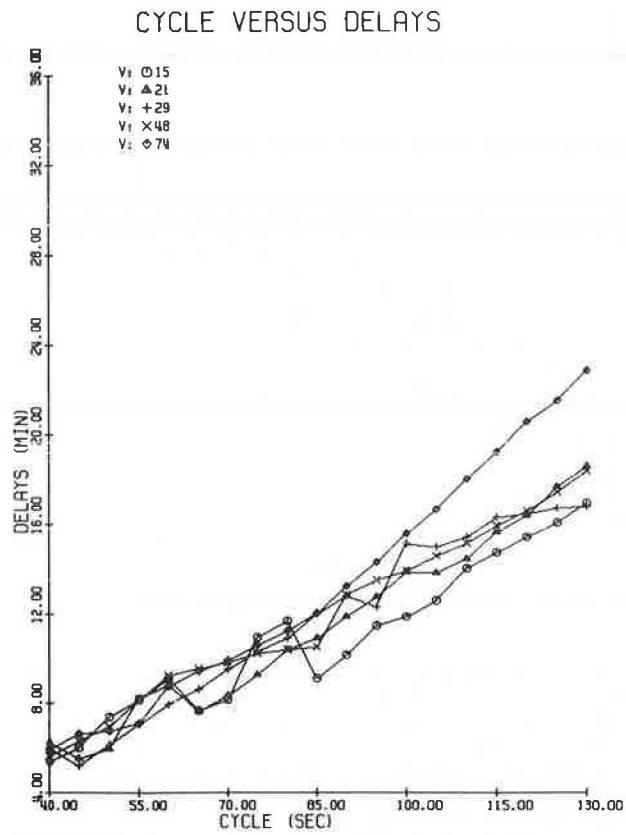

FIGURE 20 Data set 2 (Q = 850 vehicles/hr, k = 0.35).

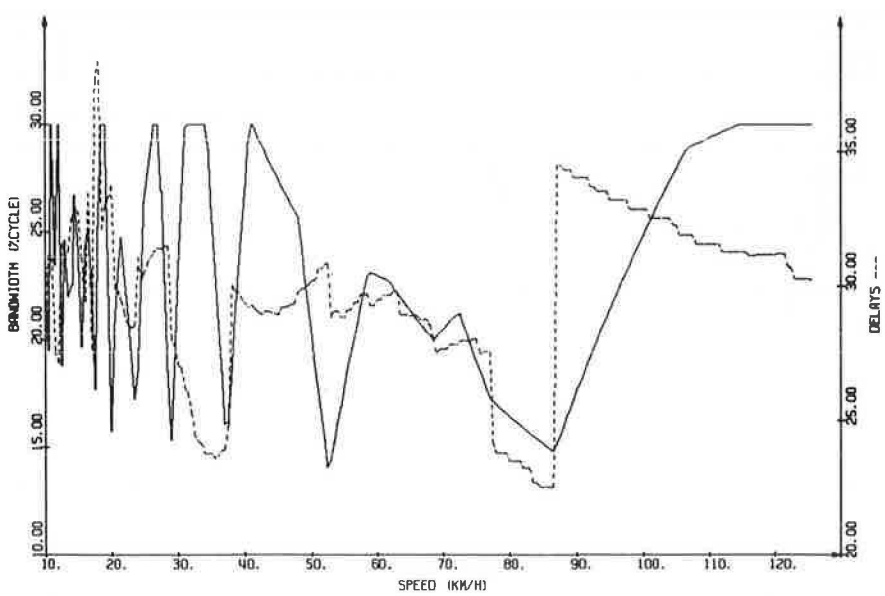

FIGURE 21 Band-speed-delay curves for data set 3 (C = 80 sec, Q = 850 vehicles/hr, k = 0.35).

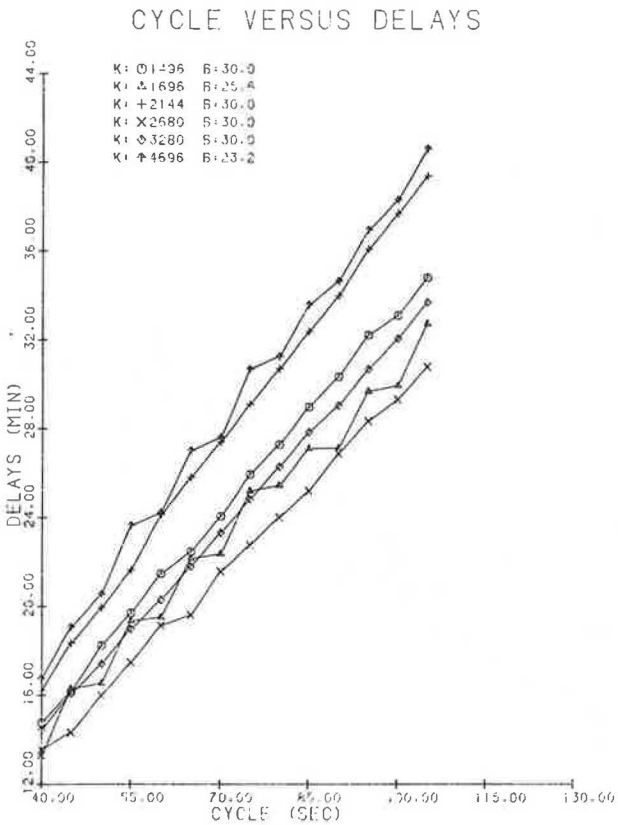

FIGURE 22 Data set 3 (Q = 850 vehicles/hr, k = 0.35).

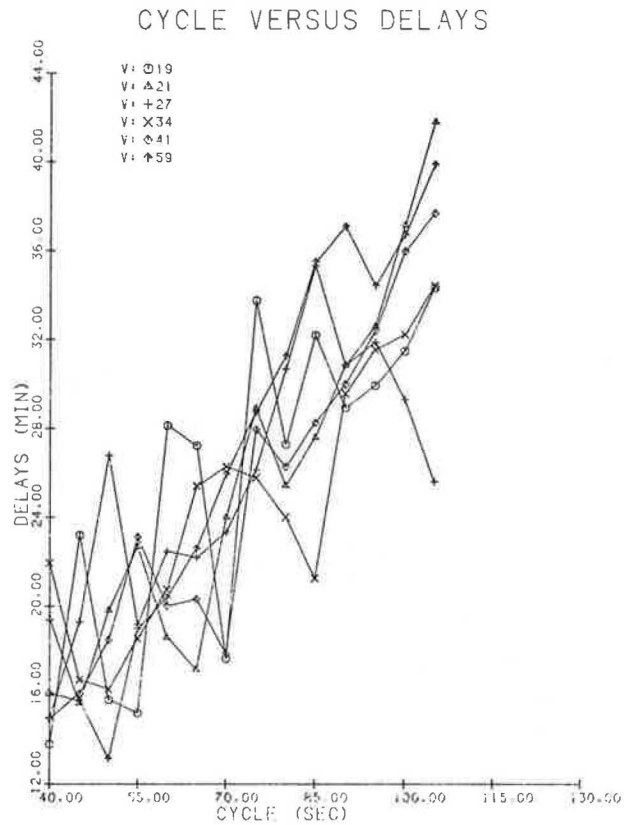

FIGURE 23 Data set 3 (Q = 850 vehicles/hr, k = 0.35).

1. The cycle length and the speed can vary between a minimum and a maximum value.
2. The cycle length is fixed, but the speed can vary between a minimum and a maximum value.
3. The cycle length can vary between a minimum and a maximum value, but the speed is fixed.

The choice of a good offset, speed, and cycle length can be done with the graphic output of the program. This is illustrated by using the example of data set 4 at Q = 850 vehicles per hour with a platoon dispersion factor of k = 0.35.

1. Let 40 < V < 55 and 55 < C < 80. Figure 27 shows high delays in the region of acceptable speeds at a cycle length of 80 sec, but there is an interesting band at V = 35.6 km/h with B = 23.8 and a delay of 33.44, the offset scheme being 100. Be-

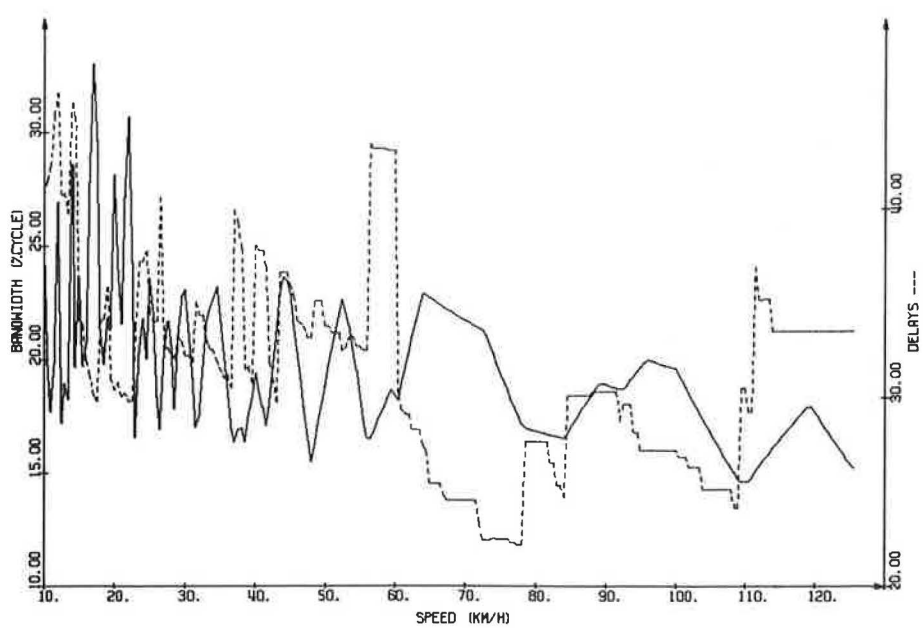

FIGURE 24 Band-speed-delay curves for data set 5 (C = 80 sec, Q = 850 vehicles/hr, k = 0.35).

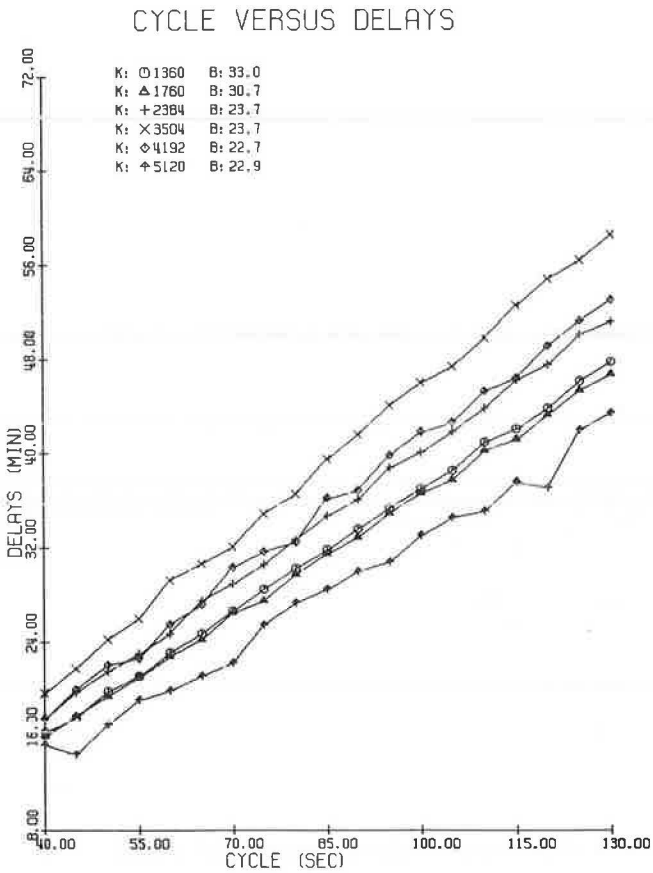

FIGURE 25 Data set 5 (Q = 850 vehicles/hr, k = 0.35).

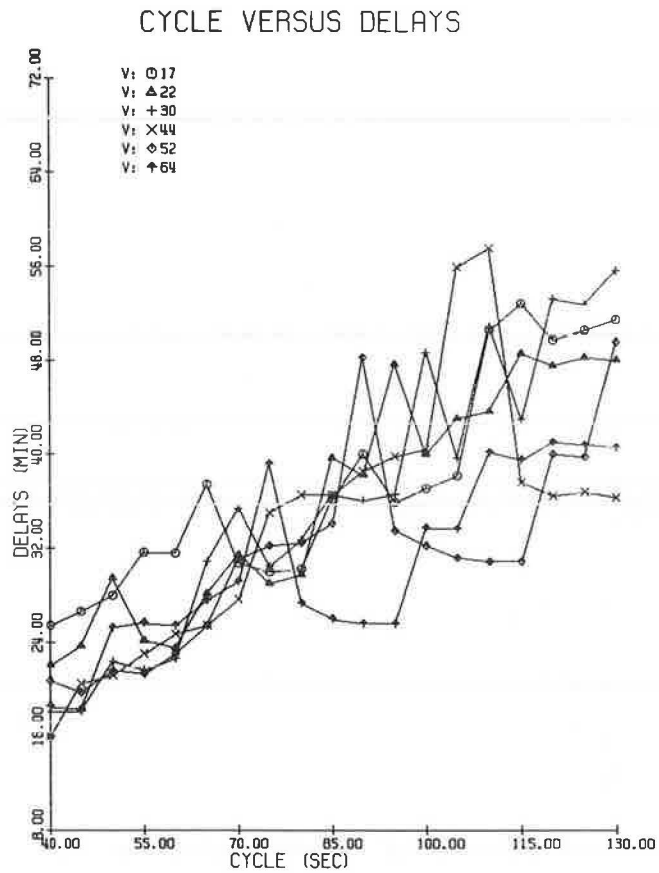

FIGURE 26 Data set 5 (Q = 850 vehicles/hr, k = 0.35).

cause the speed is too low, Figure 28 is used. This indicates that for the same value of K = V • C = 35.6 • 80 = 2,848, delays can be reduced by shortening the cycle length. The smallest allowable cycle would give the smallest delay. The solution would be C = 55 sec, speed = 51.78, delay = 32.62, and offset scheme 100.

2. Let C = 60 sec and 40 < V < 60. This corresponds to 2,400 < K < 3,600, which represents, at a cycle of 80 sec, a range of speeds of 30 < V < 45. Figure 27 would indicate the best speed of 35.6 km/h at 80-sec cycle length. The best choice would be C = 60 sec, V = 47.5 km/h, delay = 26.23, and offset scheme 100.

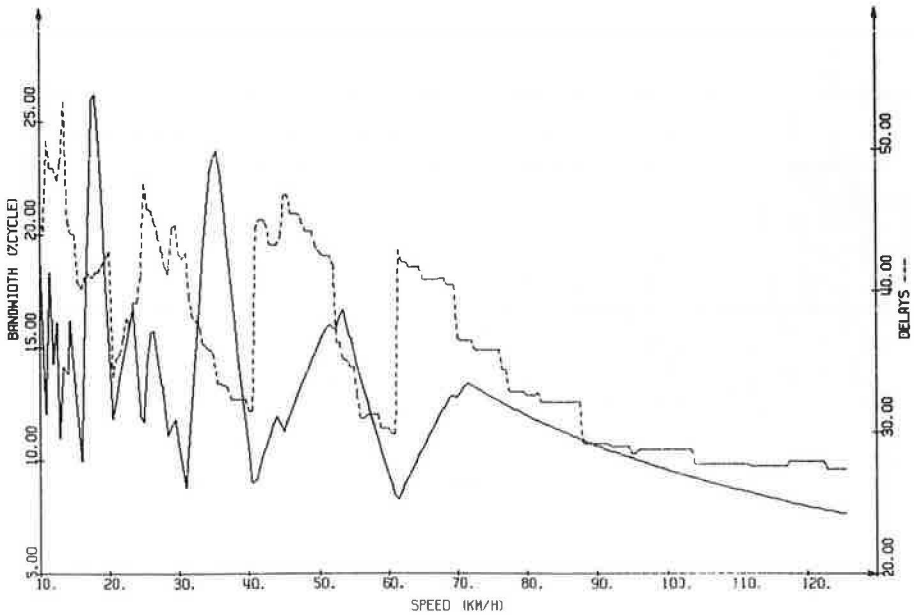

FIGURE 27 Band-speed-delay curves for data set 4 (C = 80 sec, Q = 850 vehicles/hr, k = 0.35).

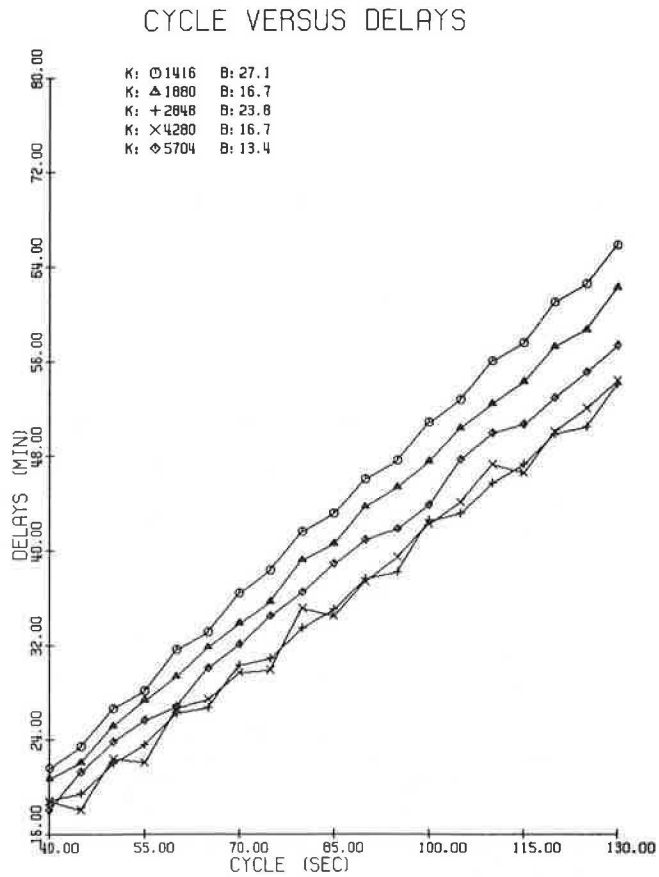

FIGURE 28 Data set 4 (Q = 850 vehicles/hr, k = 0.35).

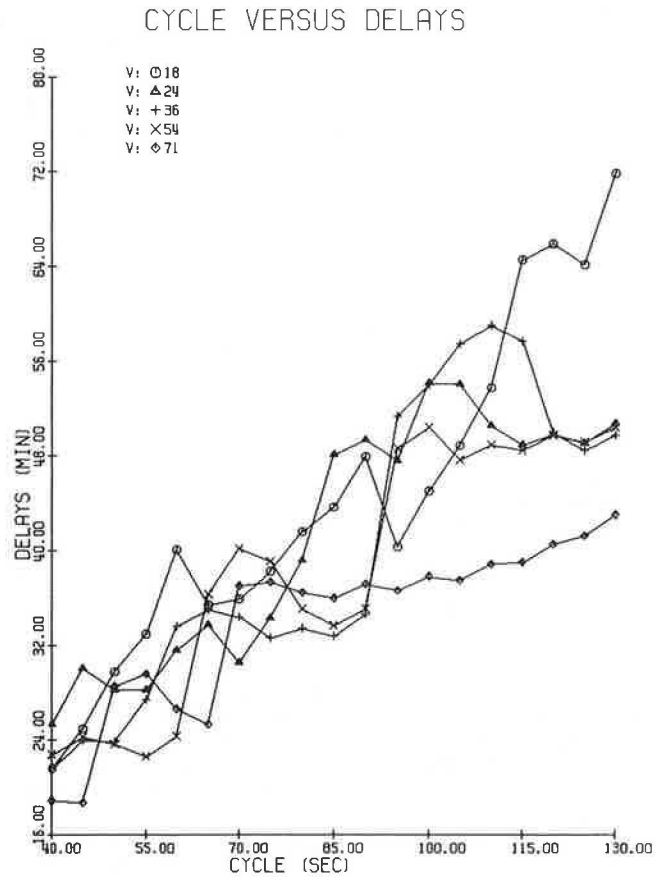

FIGURE 29 Data set 4 (Q = 850 vehicles/hr, k = 0.35).

3. Let V = 54 km/h and 80 < C < 100. Follow the curve in Figure 29 for a speed of 54 km/h. The best cycle length would be 85 sec with a band of 12.74 percent and a delay of 33.71. At a cycle of 80 sec the bandwidth would be 16.6 percent and the delays would be 35.15. The offset scheme is 31.

It would be interesting for this kind of decision to introduce an objective function that would evaluate the trade-off between a gain in delay against a loss in bandwidth.

CONCLUSIONS

Experiments with different data sets indicate that there is a clear relationship between delays and bandwidths in the case of an artery with simple geometry and no platoon dispersion. Delays are smaller as bands increase. This relation holds only for low degrees of saturation (in this case around 0.40). In more general situations of geometry and with platoon dispersion, there is no strong indication of a clear relationship between bandwidth and delay, because this depends also on speeds and offset schemes. However, the output of the program allows the identification of offsets that ensure low delays for large bands. It can be concluded from analysis of all five data sets that there is a clear relationship among K, the delay, and cycle lengths, which indicates that shorter cycle lengths cause less delay to the traffic on the artery. There is also a relation between cycle length and delay for a given speed, with the delays generally decreasing as cycle lengths are shorter.

The proposed combined method of arterial coordination analysis can be useful to the practitioner because it allows cycle lengths, speeds, and offsets to be chosen to give minimum delays while retaining a band of reasonable width. The procedure also allows the determination of the cycle length that maximizes the bandwidth for a given speed, and the speed that produces a maximum bandwidth for a given cycle length can also be found. The use of this combined approach would favor the retention of the psychological advantages of the bandwidth approach for the driver, while being, at the same time, more efficient in terms of delays. Further work will be devoted to a deeper analysis of the relationship between bandwidths and delays, and to a more detailed study of the relationships between the intervening variables and the number of stops encountered on the artery.

Discussion

Edmond C.P. Chang*

The basic issues and considerations between maximum bandwidth and minimum delay approaches to arterial signal coordination under two-phase signal operations were discussed in the paper by Baass and Allard. The procedure developed allows an engineer primarily to "determine the cycle length that maxi-

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mizes the bandwidth for a given speed and to find the speed that produces a maximum bandwidth for a given cycle length." Graphic-type comparisons were used to study the interrelationships among cycle, speed, progression bandwidth, and systemwide delay measurement. The experiment proved three basic hypotheses.

1. A larger progression band would cause fewer delays and stops than a smaller one in low saturation; that means no platoon dispersion, constant platoon speed, and fewer vehicles entering the artery from the side street.
2. In a large sample of arteries and a wide range of cycles, delay would increase on an artery when cycle length increases.
3. At a given speed, a shorter cycle length is preferable in considering delay.

Also described in the paper is the theoretical development of a computer program based on combining bandwidth maximization and disutility minimization. The results of this computer program, applied to five arterial data sets, were discussed. However, more clarification of the following topics would contribute to a better understanding of this paper.

1. It is not clear, at least at the beginning of the paper, that the application and discussion of this paper are focused on the two-phase arterial traffic signal coordination of a low-volume, low-speed, and short-distance-spaced urban grid-type network.
2. Interesting discussions in this paper introduce the possible trade-off analysis between the relationships of increase of delay and the decrease of progression bandwidth with respect to the increase of cycle length. A similar type analysis that would also be beneficial is to consider the decrease of delay versus the traffic volume difference in both travel directions in an arterial street system.
3. Many of the negative effects of platoon dispersion on delay were mentioned, but far fewer were discussed on the benefit of increased arrival traffic in a more uniform platoon and safer operation caused by maximum progression.
4. Because this paper emphasized the two-phase traffic signal, it simplified the important impact of variable phasing sequence in optimizing traffic signal operations and in reducing the total system delay.
5. A search of the optimal solution for all combinations of cycle and speed is not practical and is unnecessary if the computation algorithm starts with a satisfactory engineering solution.
6. At the beginning of the paper the disutility function of applying a TRANSYT-type macroscopic simulation model was investigated. However, it is not clear how the result, using the algorithm developed by the authors, would differ from that using the algorithm in TRANSYT under the specific coded platoon dispersion factor (PDF) and the stop penalty factor (SPF).

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Improved Graphic Techniques in Signal Progression

CHARLES E. WALLACE and KENNETH G. COURAGE

ABSTRACT

The results of several research studies into graphic representations of traffic signal system settings and traffic flow are presented. The research began with the fundamental time-space diagram, which is universally understood by traffic engineers, and also with flow diagrams from the TRANSYT model. Three specific departures from these basic techniques are presented, which include time-location diagrams, forward progression opportunities, and platoon progression diagrams. All of these techniques apply to linear arterial systems or subsystems. Another graphic technique that applies to a network of coordinated signals is then discussed: signalized network animated graphics. Interpretation of signal timing-optimization strategies, namely maximal bandwidth optimization, is discussed using the platoon progression diagram technique. This analysis demonstrates the pitfalls of the maximal bandwidth approach, thus demonstrating the power of the analysis technique.

Traffic engineers have traditionally analyzed the quality of traffic flow in coordinated traffic signal systems by either direct field measurement or by the use of off-line analysis techniques. Field studies are typically the superior approach to such analysis; however, the alternatives that may be evaluated by field studies are limited to those that the traffic engineering agency can actually install in the system. The field study approach is not conducive to design because of this limitation.

Off-line computational techniques support a wide range of designs and design strategies without requiring field implementation and evaluation studies. Although the assessment of traffic flow using such techniques is limited by the assumptions and limitations of the selected technique, their use can nonetheless dramatically increase the productivity of the traffic engineering agency.

Off-line techniques generally consist of numerical estimates of the pertinent measures of effectiveness (MOEs) and graphical representations. Numerical estimates of MOEs are typically accomplished by deterministic, or analytical, techniques; by simulation of traffic flow; or by a combination of these two. Such techniques vary considerably in their realism or accuracy, depending on the theoretical basis of the estimates; but, more significantly, they are simply numbers (e.g., delay, number of stops, fuel consumption, bandwidth) that do not offer any visual perception to the analyst as to the quality of traffic flow.

On the other hand, graphical techniques do give a picture of the perceived quality of traffic flow. Although it is true that, ultimately, MOE estimates (or even field measures) are used to quantify improvements, the graphic presentations can be of great assistance in assessing the quality of traffic flow as part of the design decision process.

A number of graphical representations of traffic signal timing and traffic flow that can be extremely useful analysis and evaluation tools for the practicing traffic engineer are presented here. First, the time-space diagram, which is universally known and used by traffic engineers, is presented, and later three specific departures that enhance the utility of this familiar graphical technique are discussed. A second existing technique for illustrating simulated traffic flows is also reviewed.

TIME-SPACE DIAGRAMS

For coordinated systems, the classical graphic presentation of traffic signal timings is the time-space diagram (TSD). A TSD, as shown in Figure 1, illustrates the relationship of a series of traffic signals in a coordinated system by showing the signal timing on the artery and the offset relationships. When the through bands are drawn, the slopes of the bands illustrate the desired speed(s) of the through bands. This is the inherent use of TSDs: to illustrate the perceived progression through a system of traffic signals.

TSDs have been produced manually for more than 50 years. Until recently, they were virtually the only method of optimizing progression on arterial routes. Given the complexity of the task, it is logical that computerized techniques would replace the manual analysis. This has, in fact, happened gradually over the past 20 years.

Computer models such as PASSER II 80 (1), MAXBAND (2), TRANSYT-7F (3), and SIGOP II (4) are among the more popular models that produce TSDs. The first two models have as their explicit objective function the maximization of through bandwidth.

The TSD presents a gross oversimplification of the traffic flow process. Its primary disadvantage as an analysis tool is that no consideration is given to the actual traffic demand. As a result, time-space or so-called maximal bandwidth-based designs may result in apparently satisfactory green bands, but in reality traffic would only be progressed into the rear of standing queues.

The usefulness of TSDs is also practically limited to linear arterials. Several attempts have been made to develop three-dimensional TSDs, but this practice is extremely laborious. Furthermore, three-dimensional TSDs are difficult to interpret. Thus the usefulness, and particularly the flexibility, of standard TSDs for networks is limited. TSDs may also be extremely lengthy for long arterials.

TRAFFIC FLOW PATTERNS

The concept of using traffic flow distributions to simulate traffic flow was introduced by Robertson (5), who described traffic flow as falling into two basic patterns, or profiles (simplified here):

1. The arrival pattern, which is the periodic flow rate of traffic arriving at a reference point on a street, which is usually the stop line; and
2. The departure pattern, which is the periodic rate of flow departing the stop line, subject to the signal display facing the traffic. [If a queue exists at the start of effective green (i.e., green start plus lost time), it departs at the maximum

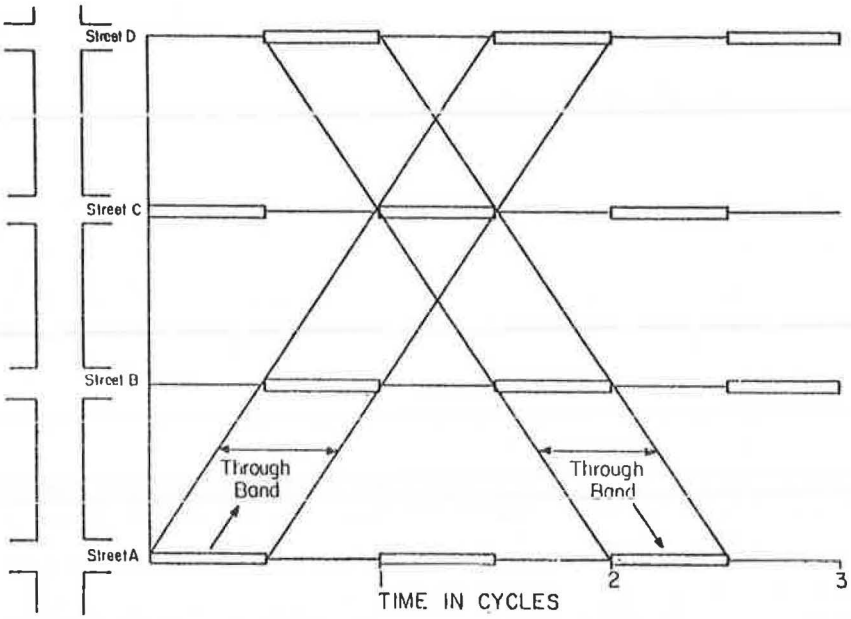


FIGURE 1 Typical TSD.

rate, or saturation flow and arrivals continue to add to the back of the queue. Once the queue has dissipated, departures equal arrivals and are not delayed.]

These patterns are illustrated in Figure 2.

This concept has been implemented in the Traffic Network Study Tool (TRANSYT) model (5), which has

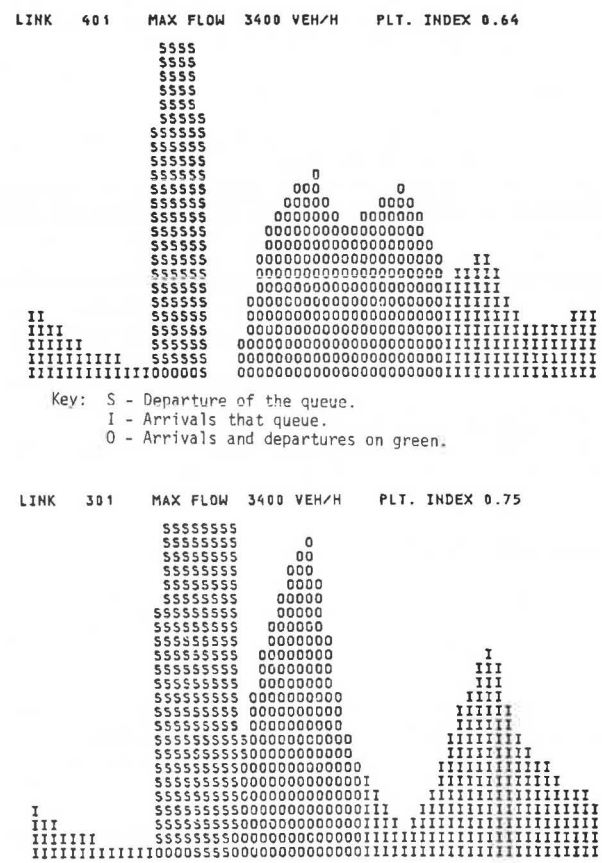


FIGURE 2 Typical flow pattern diagram.

further evolved to versions TRANSYT-7F (3), which is used in the United States, and TRANSYT-8 (6), which is available from the Transport and Road Research Laboratory.

The flow profile represents a snapshot of traffic flow on a single approach, or link, for one signal cycle; thus it represents the average traffic behavior that might exist over a longer period of, for example, 1 hr. This is the distinct advantage of this form of graphic display, because the simulated traffic flows, arrivals, and departures are actually shown. When TRANSYT is optimized to minimize stops and delay (or excess fuel consumption), it is usually trying to position the green interval on every link to clear the queues before the arrival of the moving platoons, provided the proper stop penalty is used. The same purpose is served by the queue clearance options in the PASSER II 80 and MAXBAND models, but in TRANSYT the adjustment is made explicitly.

Of the available graphic techniques, these profiles best visualize the quality of traffic flow. For example, Figure 2 shows two links, where (a) the traffic platoon arrives after the queue has dissipated, and (b) the platoon arrives just at the departure of the queue. In both cases there is minimal delay to the primary platoons, but if this offset relationship was plotted on a TSD, the band would appear to be too fast.

The disadvantages of this technique are as follows.

1. Some traffic engineers find the profiles difficult to interpret because the vertical scale is a flow rate. (The horizontal scale is time in steps of the cycle.)
2. The profiles are at the stop line at each link; thus the system interrelationships are difficult to visualize, although they can be aligned to show traffic progression down (southbound) one side of the computer output and up (northbound) the other side.
3. Similarly, the signal timing parameters are not readily apparent, and optimization by TRANSYT will typically result in offsets that appear to be too early because of the queue clearances noted previously. Engineers accustomed to TSDs do not visual-

ize the progression because the apparent speed is too fast.

With regard to the last point, it appears that the concept of through bands representing progression is misleading, and a better interpretation of satisfactory progression is the uninterrupted propagation of platoons. This is discussed in more detail later.

INNOVATIVE TECHNIQUES

In recent years several new concepts of traffic progression and the graphical representation thereof have emerged, particularly for coordinated systems. These are described in the following subsections.

Time-Location Diagrams

As mentioned earlier, TSDs can be lengthy for long arterials. This not only wastes paper, but also causes problems with report reproduction. More significantly, TSDs generally do not fit on video monitors, and the increasing use of computerized tools would render this a disadvantage.

The progression speed (i.e., the slope of the bands) is usually given and is therefore of less concern than the bandwidth, which is the MOE. A TSD can be easily modified by correcting the plotted offset to account for travel time, such that the slope of the band becomes zero at the desired speed. The plot can then be rotated from link to link so that a horizontal line (assuming the vertical axis is time and distance is on the horizontal) would represent the desired speed, or zero slope. The distance between intersections is no longer meaningful, so the diagram can be collapsed. Thus the vertical axis remains time, but only the relative order of intersections is important on the horizontal scale.

When this is done for both directions of travel, two so-called time-location diagrams (TLDs) can be plotted next to one another. The TLD technique was first reported by Wallace and Courage (7), and an example is shown in Figure 3, which was installed in a modified version of TRANSYT-6C (8). This TLD represents a design based on PASSER II's bandwidth optimization.

More significantly, the TLD concept has been incorporated into the arterial analysis package (AAP) (9) microcomputer routine SPAN (10). By using either keyboard entry on an APPLE computer or transfer of special outputs of the AAP from the mainframe computer to the APPLE, a TLD can be plotted directly on the monitor, as shown in Figure 4.

By observing the TLD and recalling that zero slope represents the desired speeds, progression in the classical sense is visualized as a horizontal tunnel of green through all the intersections for both directions. This diagram can be manipulated to change offsets to improve progression if desired.

The TLD is a simpler tool to use than the TSD because it is compact and the quality of progression

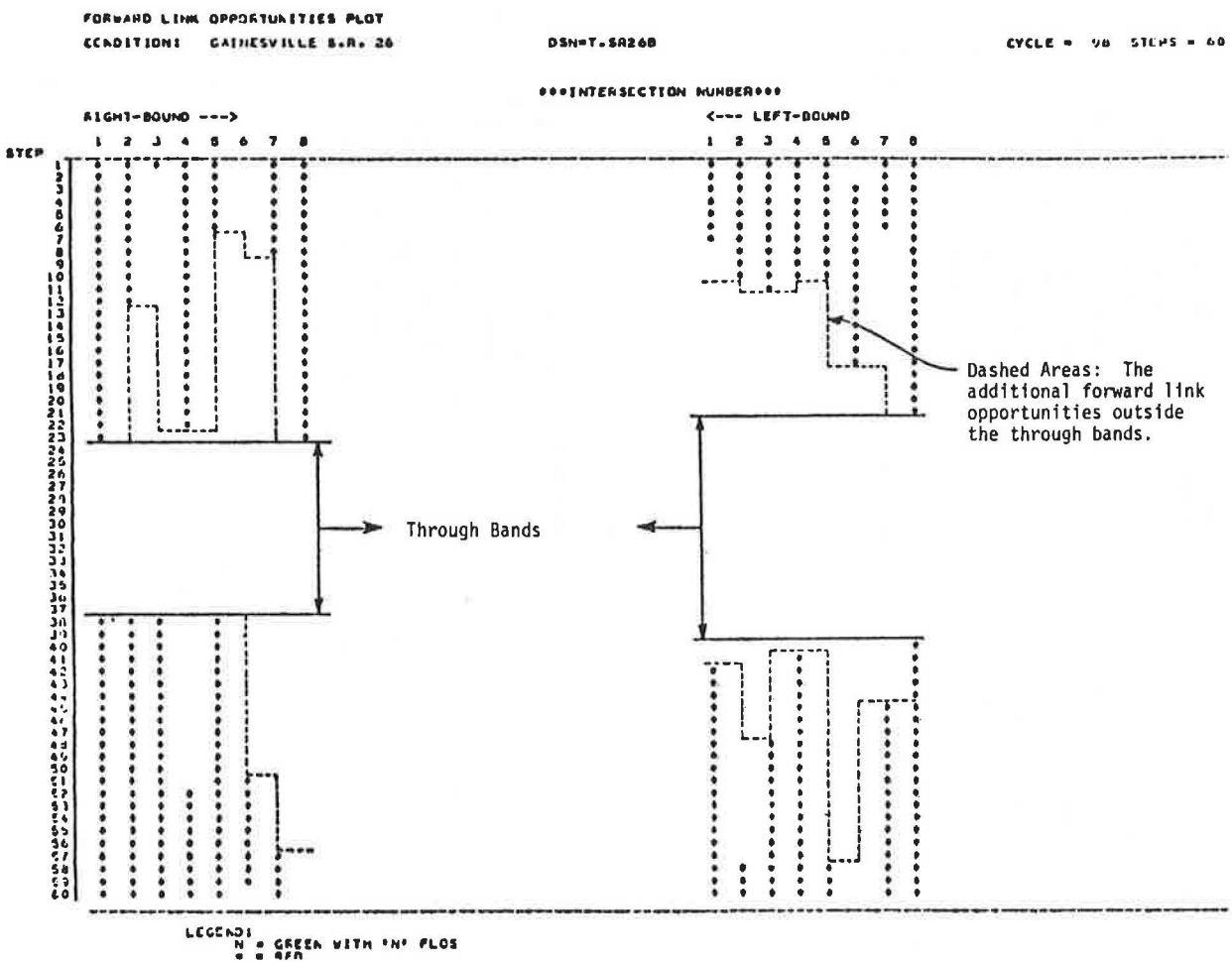


FIGURE 3 Printed TLD.

is much more apparent visually. It is particularly useful for visualizing progression throughout a portion of the system and for identifying critical signal locations.

Forward Progression Opportunities

The concept of the TLD was extended by the authors to improve on the basic concept of traffic signal design based on time-space relationships, particularly in computerized optimization models. Maximal bandwidth models such as PASSER II 80 and MAXBAND are only concerned with through bands. Some intersections are noncritical, and their resulting offsets will not be assigned with any specific objective in mind. Furthermore, there are often progression opportunities that do not exist over the entire length of an artery but do exist in short sections where progression could be beneficial.

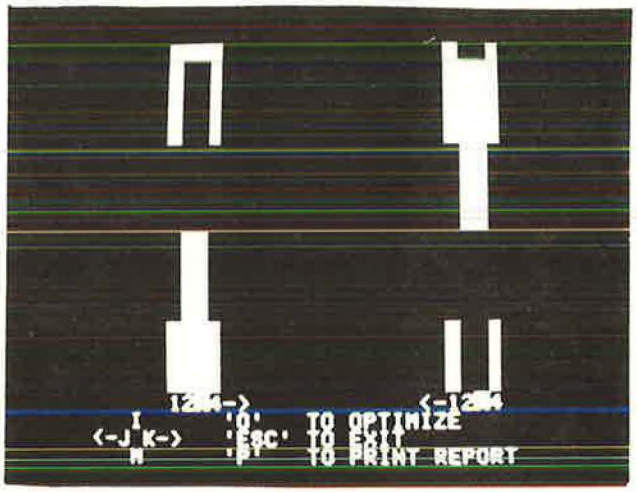


FIGURE 4 Computer plot of TLD.

The concept of forward progression opportunities (PROS) was developed to overcome this deficiency (7,8). A forward progression opportunity is simply the opportunity presented to the motorist arriving during various times in the cycle to travel forward on one link of an arterial system without being stopped by a signal at either end of the link. PROS can thus be quantified by examining the progression opportunities periodically throughout the cycle and summing them. Signal timing optimization can likewise be based on maximizing PROS. The difference between a maximal bandwidth design and a PROS design on the same facility (optimizing offsets only) clearly illustrates the advantage of the PROS approach, as seen in Figure 5 (i.e., compare the dashed areas representing PROS with the similar areas in Figure 3).

The PROS concept has been implemented in TRANSYT-6C and has been proposed as an enhancement to TRANSYT-7F and the AAP.

Platoon Progression Diagram

In the earlier section on existing graphics tech-

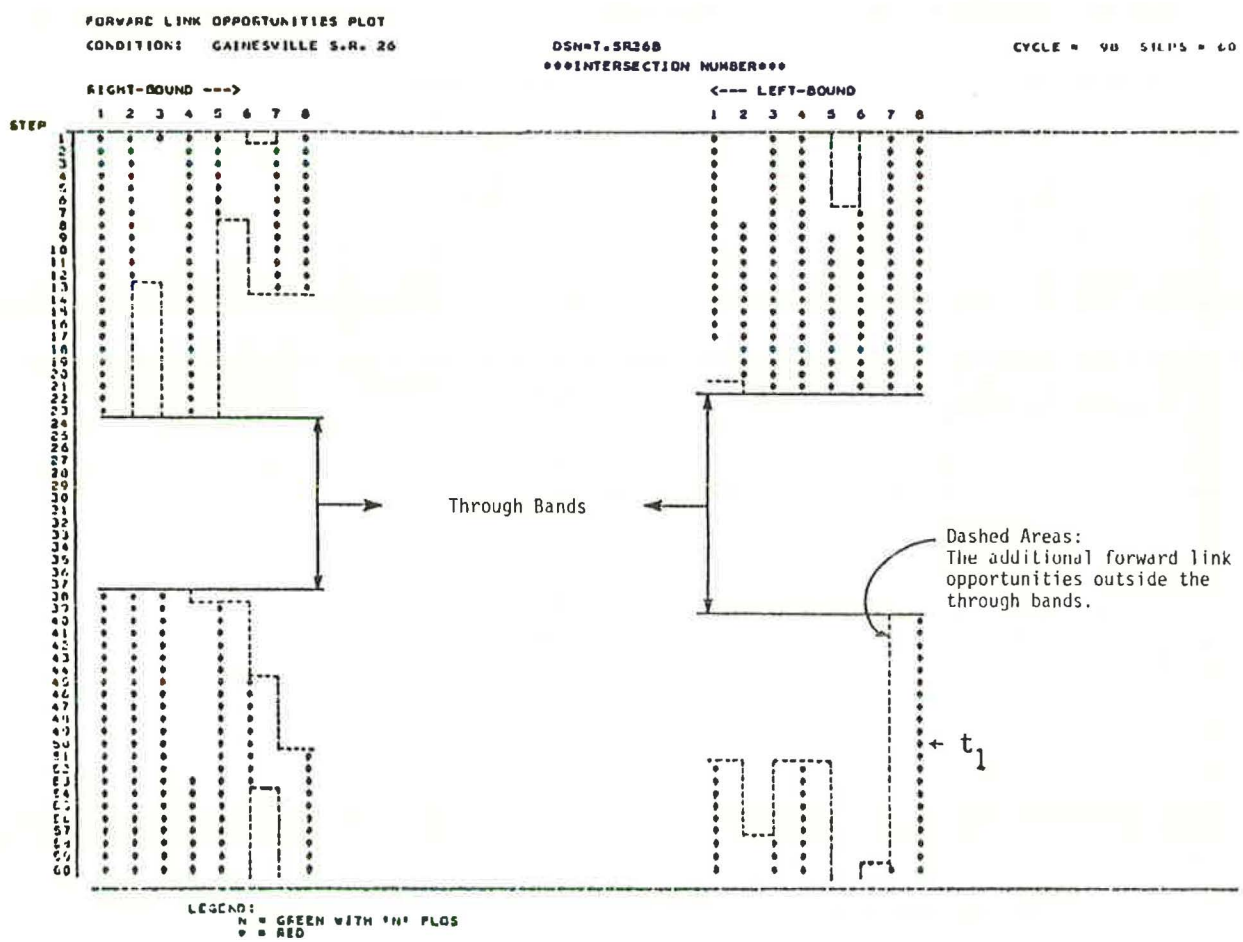


FIGURE 5 TLD from a PROS optimization.

niques, TSDs and platoon profiles were introduced, and the relative advantages and disadvantages of each were given. A review of these techniques suggests that the disadvantages of one approach were advantages of the other to some extent. The logical question is, Why not combine the two?

Considering that both require two dimensions (i.e., time versus distance and flow rate versus time), a total of three dimensions are needed; therefore, a method is required to represent the third dimension in a two-dimensional graphic to display on a monitor or to print. The approach taken by the authors was to plot a standard TSD in two dimensions and express the platoon profiles as a density function. A platoon profile can be sliced into several relative levels of flow rate as shown in Figure 6, where the higher the flow rate, the denser the area to be plotted.

A microcomputer program was developed to accept various output data readily available in the TRANSYT model (which includes timings, saturation flows, platoon dispersion factors, and the departure patterns on each link), and to use these data to produce a platoon progression diagram (PPD) display. The program applies TRANSYT's platoon-dispersion model every 50 ft along the artery and converts the propagated profile to a density function, normalized to the maximum flow rate, which is then plotted. A simplified PPD is shown in Figure 7. The dark areas departing from upstream intersections represent flow

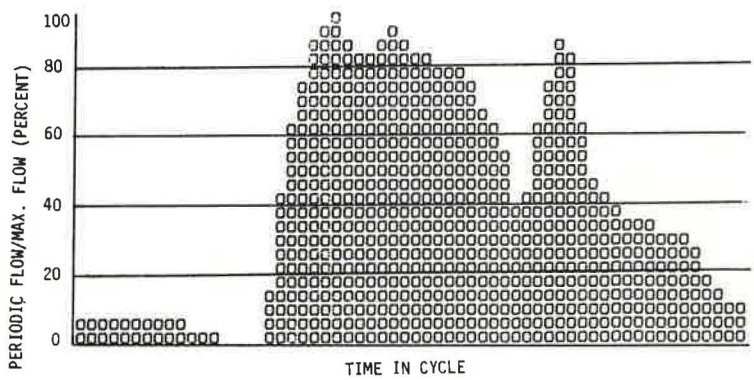
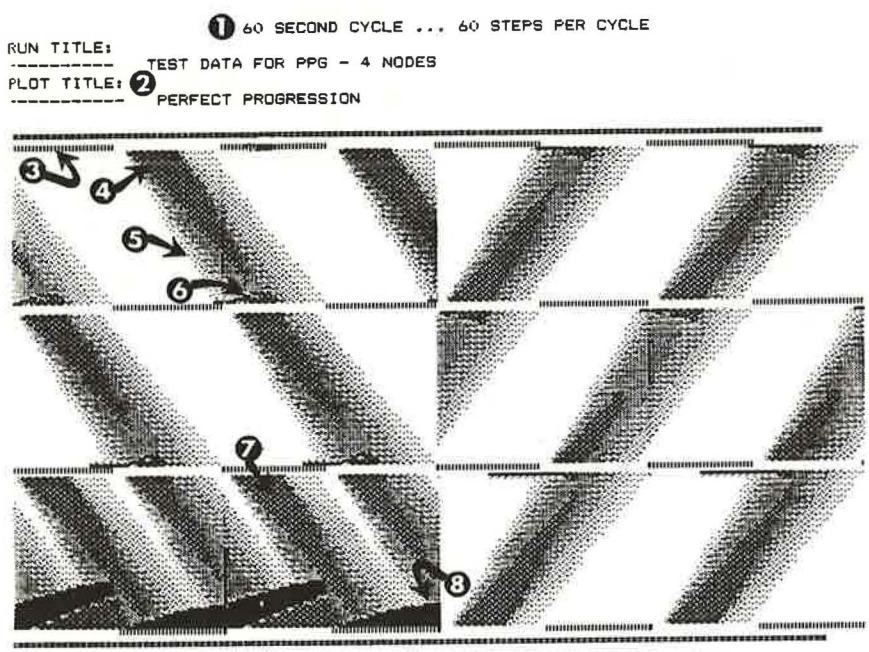


FIGURE 6 Flow profile illustrating density function.



1 The cycle length and step size are TRANSYT inputs.

2 The titles are also entered into TRANSYT.

3 SIGNAL STATUS - (red or green) is shown just like a time-space diagram.

4 The platoon is saturated as it leaves the intersection.

5 The platoon spreads out as it progresses down the street.

6 A small queue builds up here because a few vehicles have been stopped on the red.

7 This platoon results from turning traffic entering the link on the cross street green.

8 This large queue builds up because the link volumes are near capacity.

FIGURE 7 Typical PPD.

at or near the maximum, whereas the lighter areas are flows of lesser magnitude, perhaps undelayed arrivals. As the platoon travels downstream it disperses, so that the length of time to which the highest flow rate applies will decrease; thus the dark areas become narrower and eventually disappear. The lighter areas appear to diverge, again representing the physical lengthening of the platoon in time as it disperses. The PPD therefore graphically shows the platoon behavior over the entire block or link length. Cross-street traffic is shown as platoons departing upstream in the red of the arterial.

As the traffic approaches the downstream intersection and if it arrives on red, it builds a queue as shown in Figure 7. The queuing model is based on input-output calculations and assumed vehicle lengths. Platoons that are not delayed pass through on green.

This graphic has the following advantages:

1. The best features of TSDs and platoon profiles are combined;
2. Traffic progression (as opposed to green time alone) is clearly represented;
3. The effect of queuing is clearly shown;
4. The point made earlier (i.e., the desirability of clearing the queue before the arrival of the platoon) is made obvious; and
5. The graphic is easier to interpret than flow profiles, particularly in terms of system performance, and it shows the pitfalls of a straight bandwidth approach.

The outputs to this program are directly available from Release 3 of TRANSYT-7F. The PPD shown in Figure 7 was produced by the BITE (11) program.

Signalized Network Animated Graphics

It was also noted earlier that TSDs for networks are extremely difficult to construct and perhaps even more difficult to interpret. The problem again is the need for three dimensions where only two are available on a monitor or on a printed page. One approach to solving this dilemma is to actually use three dimensions, where time is the third dimension. Imagine a TSD on a linear route with the through band drawn. If a slice or cross section of the band is drawn on the distance axis every short increment of time, the physical location of the green band can be located on the route as a function of time. If the green band was superimposed over the street itself, and subsequent frames of such a picture were viewed in sync, the band would appear to move along the route.

The same representation can also be shown on a network of streets, and the bands would then be moving along all streets in the appropriate directions. This concept was first implemented by Courage (12) by using a computer output microfilmer, an expensive machine that has now become obsolete.

The concept remains quite compelling, and with the power of microcomputers and computer graphics, the authors have currently developed a microcomputer-based model called signalized network animated graphics (SNAG), which will be economically viable for widespread use. This model uses an IBM Personal Computer with medium-resolution color graphics. Sample frames showing the display at three points in the cycle are presented in Figure 8.

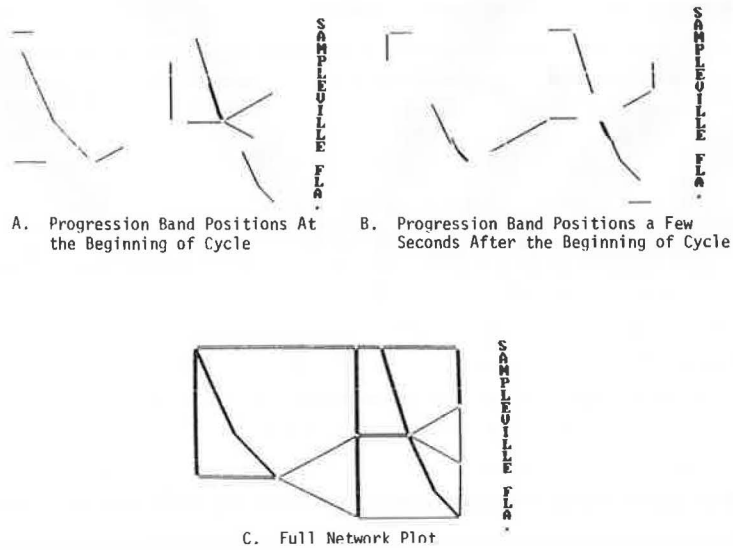
Another extremely important advantage of the SNAG animated network is its public appeal. The concept is intuitively simple to understand and can be used as a public relations tool to demonstrate to the general public and administrative officials how effective a completed or proposed traffic signal system improvement project has been or will be.

Considering the earlier comments about the usefulness, or rather limitations, of TSDs per se, a further extension of the SNAG concept will be to animate the movement of traffic platoons rather than green bands. This development should prove far more useful to the traffic engineer.

USING GRAPHICS TO INTERPRET DESIGN EFFECTIVENESS

The foregoing discussions have briefly described several commonly used and several innovative graphic tools for analyzing signal timing and traffic flow. It is noteworthy to repeat that traffic signal system timings have traditionally been based on time-space relationships by using manual methods or computer models like PASSER II 80 and MAXBAND. Recently, however, an increasing number of practitioners are shifting to strategies that optimize system efficiency, using models such as TRANSYT-7F and SIGOP III.

This shift in strategies holds merit, and an ex-



A. Progression Band Positions At the Beginning of Cycle

B. Progression Band Positions a Few Seconds After the Beginning of Cycle

C. Full Network Plot

FIGURE 8 Sample frames of three points in cycle from the SNAG program.

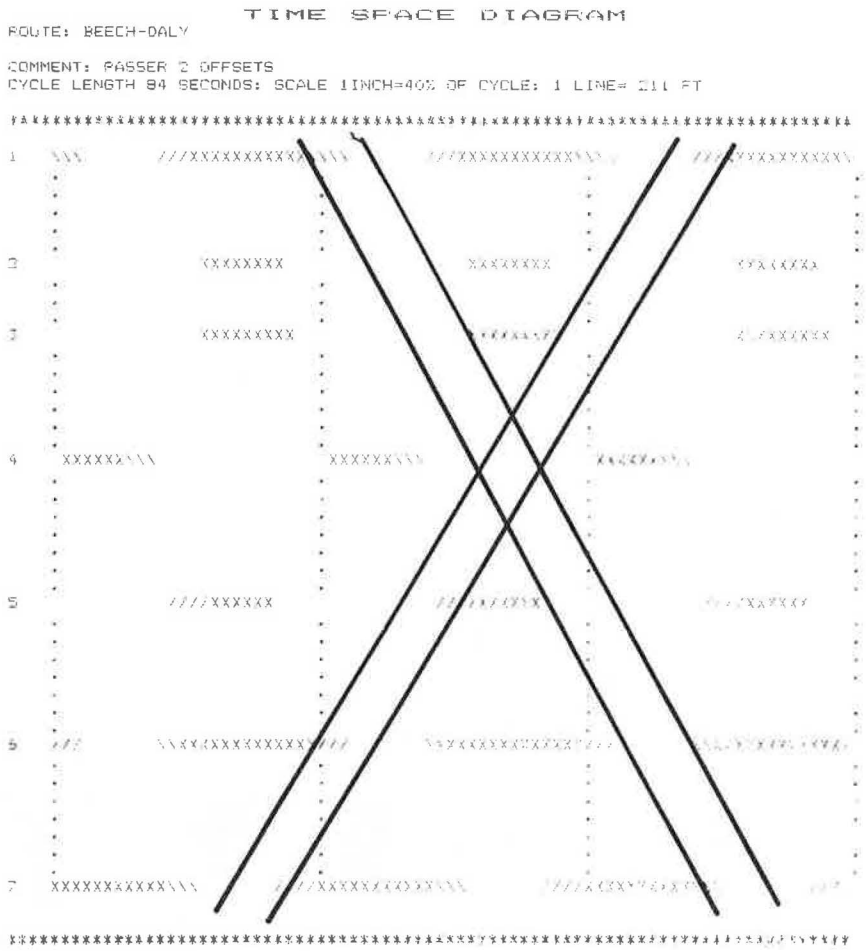


FIGURE 9 SPAN TSD of Beech-Daly system.

ample is given in this section by using the PPD concept to graphically illustrate the advantages of the system efficiency strategy.

The following example compares the TSD with the PPD and the TLD for a section of Beech-Daly Road in Detroit. The same design (PASSER II optimization) is depicted in each case. A constant speed of 45 mph is assumed throughout this section, which contains seven traffic signals.

The TSD is shown in Figure 9. This is a standard printed output from the SPAN program, auto-scaled to fit conveniently on one page. The progression bands have been added to this figure for emphasis.

The corresponding TLD is shown in Figure 10; this also is an output of the SPAN program. Again the leading and trailing edges of the progression bands have been added. Note that in this case the bands do not have the characteristic slope of the TSD because of the correction for travel time. This permits the entire route to be compressed into a much smaller space. The advantages of the compression are as follows.

1. Compatibility with the shape of the videoscreen on a microcomputer: The SPAN program displays the TLD on the screen and allows manipulation of offsets from the keyboard. This is a powerful editing feature for the design of simple arterial systems.

2. Assessment of the quality of progression: Progression throughout a portion of the system is easier to visualize on the TLD because all of the signals are immediately adjacent to each other. Critical signals that interrupt progression are also more apparent. Note, for example, how intersection 3 stands out as the critical signal for rightbound progression in Figure 10.

Some useful MOEs are also included in Figure 10. In addition to the commonly used measures of bandwidth, efficiency, and attainability, three other values are provided.

1. System offset: This is not really an MOE but simply an indication of the amount by which all offsets were shifted to center the progression bands on the page for easier interpretation.

2. Performance index: This measure indicates the total PROS, as defined earlier in this paper. It is expressed as a proportion of the cycle length. Its value is usually greater than the progression efficiency because of progression opportunities that occur throughout a portion of the system (e.g., between intersections 2 and 4, rightbound).

3. Interference: This measure indicates the proportion of time in which a vehicle released from one signal will be stopped at the next signal. It has at least an intuitive connection with safety and driver comfort. Interference is much more apparent on the TLD because of the adjusted alignment of the red intervals.

The TLD and TSD only show time relationships among the signals. As such, they are not concerned with the actual movement of traffic. As noted earlier, this is their main shortcoming. They reflect

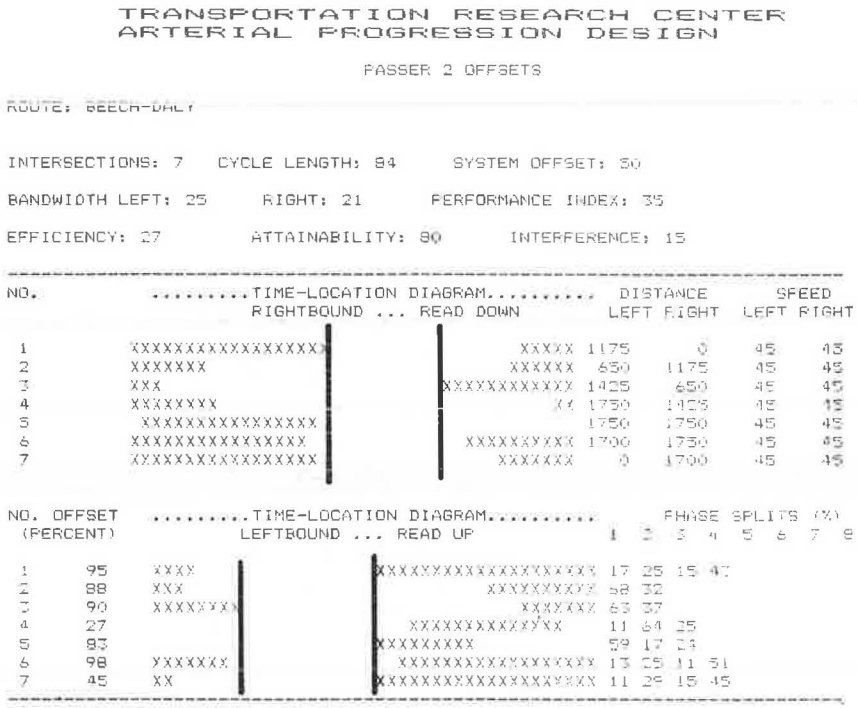


FIGURE 10 SPAN TLD of Beech-Daly system.

what the motorist can expect to encounter only under extremely light traffic conditions. As traffic volumes increase, the actual conditions will depart substantially from the ideal picture presented by the TSD and the TLD. The PPD provides the solution to this problem.

The PPD for the Beech-Daly system is shown in Figure 11. Two points are apparent:

1. Considerable movement of traffic occurs outside of the progression bands (note that the bands are added to the drawing for purposes of comparison), and
2. Queues build up within the bands at several intersections; in other words, the progression bands travel smoothly through the system, but most of the vehicles must stop.

The PPD clearly shows how the traffic is affected by the signals. It also provides some insight into the rationale behind the signal timing design produced by the TRANSYT model. The TRANSYT view of the system operation is exactly what is shown on the PPD.

CONCLUSIONS

The recent massive increase in the use of microcomputers has provided the traffic engineer with greatly enhanced graphics capabilities. An excellent example is found in signal progression design. The time-space diagram, which has been used universally for the past 50 years, is primitive and inadequate for many purposes. The graphics techniques presented in this paper extend the concept of the time-space diagram. The techniques may be implemented easily and are powerful tools in the design and analysis of traffic control systems.

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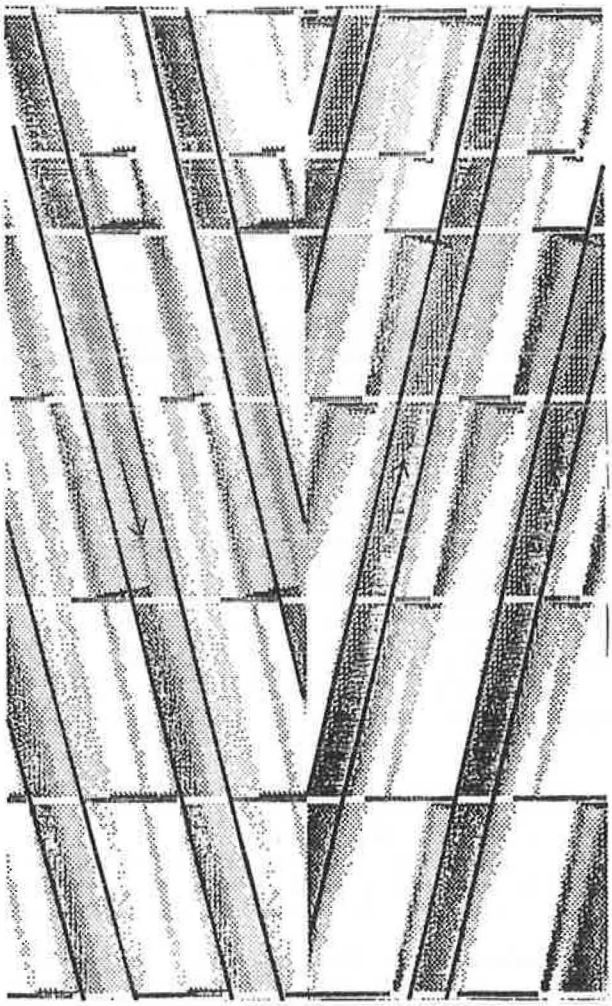

FIGURE 11 PPD of Beech-Daly system.

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Analysis of Parking in Urban Centers: Equilibrium Assignment Approach

YEHUDA J. GUR and EDWARD A. BEIMBORN

ABSTRACT

Parking policies and supply play a major role in the management of transportation systems in dense urban areas. A method for representing and analyzing parking is described. Included in the procedure are calculations of parking impedance for each destination point in a study area and the determination level of use of each parking location in the area, including illegal parking. In the model the amount of time spent looking or waiting for a parking space is an increasing function of the utilization level of the parking area. With this relationship it is possible to describe and analyze the parking process in the framework of user-optimized equilibrium assignment. All of the major factors that affect parking behavior in urban areas are accounted for, including walk to destination, parking fees, parking regulations, intensity of enforcement, and supply-demand relationships. The model and its testing in a dense section of the city of Haifa, Israel, are described. In this test case parking behavior is examined as it varies with value of walk time, parking cost, parking fines, enforcement policies, and level of travel demand.

In densely developed sections of urban areas such as the central business district (CBD), parking constitutes a large part of the total travel impedance of travelers who use private automobiles. Parking management can be an effective tool for controlling the number and nature of automobile arrivals. Such control can be realized in a number of ways, from individual changes in modal split or trip scheduling to overall changes in the level of activity of an area. Possibly one of the most important questions in the management of transportation in urban centers is the determination of a level of parking supply that encourages arrival by public transportation, while at the same time prevents loss of activity because of parking shortages. Because of the fear of potential business loss through rigid parking control, managers are unwilling to experiment with changes in parking policy as an element in the management of transportation systems.

A model that analyzes parking in dense urban areas is described. The model is designed to serve as part of a modeling system for the analysis of the impact of integrated transportation system management (TSM) strategies in city centers. The parking model has also proved to be an effective independent tool for parking design, and it will be presented here as such.

The purpose of the model is to simulate parking choice and to provide the following information:

1. Estimates of parking impedance (or disutility) while traveling to specific destinations;
2. Estimates of the use of specific parking areas;
3. Estimates of the impact of changes in parking demand, such as changes in the number of automobile arrivals, changes in parking duration, and changes in the location of new activity centers; and
4. Estimates of the impact of changes in supply, including changes in the number and location of parking spaces, changes in parking regulation and parking fees, and changes in the degree of enforcement.

In the following sections the nature of the problem, previous attempts to address the problem, and the approach used in this paper, which is based on the principles of impedance minimization and equilibrium traffic assignment, are described. Also included is a description of the model's application and test in the CBD of Haifa, Israel.

NATURE OF PROBLEM

Travelers who arrive in a city center by automobile can be typified by their (a) specific destination, (b) desired parking duration, and (c) time of arrival. Parking places include both on-street and off-street locations (both will be referred to as lots) that are typified by the number of spaces, operating rules (parking fees, parking regulations, enforcement practices, and so forth), and location. The total parking impedance includes both the impedance of parking at the lot (fee, waiting time, and so forth) as well as the walk time from the parking location to the destination.

Each parking space can potentially serve a number of arrivals (trips), especially trips with short parking duration. On the other hand, because of demand characteristics, most parking spaces are used only for a part of a day. Thus it may be necessary to analyze parking with concern to the time-of-day variations.

It is unrealistic to assume that only legal parking places are used. In congested areas there can be a significant amount of illegal parking. Illegal parking may be considered by users to be a valid parking alternative when the expected fine is weighed against high parking fees or long walks from alternative parking locations. Almost any potential parking space should be considered. In the case of illegal parking, the impedance calculations should include the level of fine and the intensity of enforcement (which determines the probability of getting caught). The rest of the enforcement process should also be considered, including the effectiveness of the fine collection process and the dampening effect of long delays in collection.

Because of the highly dynamic nature of parking in urban centers, a strict definition of capacity of parking lots is not easily obtained. In most cases a traveler arriving to a full lot is willing to wait a reasonable time for a space to be vacated. Similarly, even when lots are not completely full, travelers spend some time looking for a space. It can be claimed that the impedance of parking increases (quite quickly) with the increased use of the lot; it is quite realistic to describe parking delay in terms of increasing volume-delay or occupancy-delay functions.

In assigning parkers to lots, it will be assumed that travelers try to minimize their disutility. A promising principle for this purpose is the user-optimized equilibrium assignment model, which is based on the principle that travelers cannot improve their parking disutility by unilaterally changing their assigned lot.

PREVIOUS WORK

The study of parking problems has received extensive attention in the form of individual parking analyses and studies at specific locations, yet the number of efforts to model parking behavior has been limited.

In practically every travel demand model, parking impedance enters as a component in the disutility calculations, but there is only scant reporting on the method in which it is estimated. It appears that, for the description of existing conditions (e.g., for model calibration), a reasonable parking location can be assigned to each traveler for an approximate estimation of parking impedance. However, such a method is not valid when a major change in modal split is expected or when there is a need to estimate the effect of changes in parking supply or policy.

Early work done by Ellis and Rassam (1) suggests a basic approach to modeling parking behavior; they propose the use of an optimization technique to allocate parkers to parking locations. The technique follows the basic transportation problem from operations research, where the total disutility of parking for all persons is minimized, subject to supply and demand constraints. In their application a linear programming approach was used. Whillock (2) also proposed a parking model of a similar structure.

A second approach that has been used in parking studies is to use a gravity model formulation to model parking location. This has been used by Bates (3), Austin (4), and Bullen (5). Under such an approach parking supply and demand are balanced, with parkers allocated on the basis of trip ends and walking distance from a parking location. A problem arises with such an approach when parking demand exceeds capacity of a given location. When this occurs the excess demand must be iteratively reassigned to other locations.

THE MODEL

Two key elements were necessary to describe the parking process: (a) equilibrium assignment--the development of a means to assign parkers to locations in such a way as to recognize capacity limits and relative disutility of alternative locations consistent with optimized equilibrium assignment principles; and (b) parking disutility--the determination of the disutility of parking at a given location. The overall flow of the model is shown in Figure 1. Fixed components of disutility (i.e., those that do not vary with traffic volume at a given location) are first calculated. These include fees, penalty costs, and walk time to the destination. This information is then used to find the least path from each parking location to each destination point. Trips are loaded on these paths, and look times are calculated for each path. An equilibrium assignment is then used to bring supply and demand into balance. The following sections explain the components of the model.

EQUILIBRIUM ASSIGNMENT APPROACH

It appears attractive to model parking choice as a user-optimized equilibrium network assignment problem (6). The basic principle of equilibrium assignment (that no traveler can decrease his impedance by unilaterally changing a lot or, more generally, a

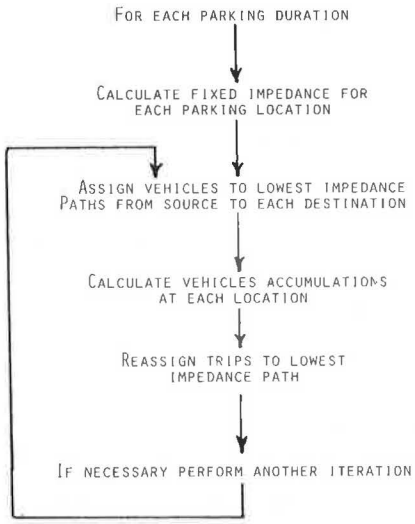

FIGURE 1 Model flowchart.

path) provides a reasonable model for the parking process. This is the approach described in this paper.

The path-finding approach is shown in Figure 2. It is assumed that all trips originate in one source (O) and go to the destination zones (D). The number of trips to each destination is known. The source is connected to the parking lots by automobile links. Each parking lot is described as a parking link, which is connected to the destinations by walk links. The coding scheme ensures that every path (from the source to a destination) includes exactly one parking lot and at least one walk link. The equilibrium assignment model distributes the parkers among the various lots, so that each parker is assigned to the lot that minimizes their total impedance, which includes both parking and walking.

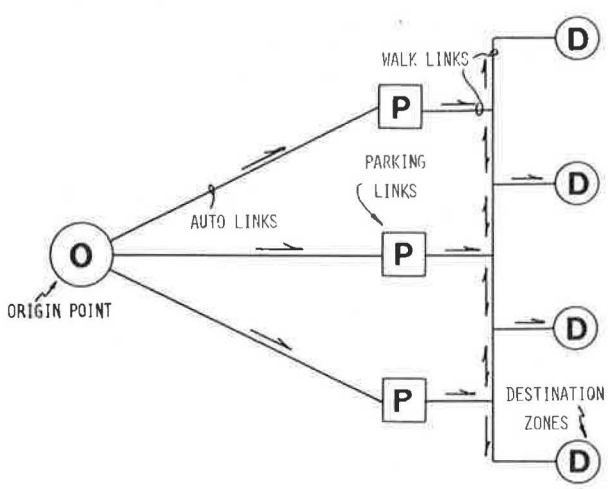

FIGURE 2 Schematic parking network.

The assignment model imposes few requirements on the data structure. Each link has a scalar impedance measure, and the impedance of a path equals the sum of the impedance on its links. Impedance of the walk and automobile links is related to their travel (or walk) time, whereas the parking links are assigned an impedance related to the cost of parking, the waiting for an open space, and the potential risk of parking fines.

In conventional equilibrium assignment modeling all links must have a monotonically increasing volume-impedance function in order to guarantee a unique solution. In parking analysis it is not necessary to relate walking or driving impedances to volume changes due to parking lot choices; only the parking impedances change with demand. This is not a serious problem because each path includes a parking link whose impedance is volume dependent. It is assumed that all automobile links have the same impedance (i.e., that variations in driving time do not affect the choice of a parking lot). In some cases this could be inaccurate. This can be dealt with by replacing the demand source, by actual trip origins (at entry points to the CBD), by describing the demand as a trip table, and by coding the highway network. In the present application this coding scheme was not necessary.

For this study there was a need to extend the classical equilibrium assignment model. There was a need to treat travelers with varying parking durations. Obviously, the parking impedance, and hence the choice of a lot, depend on the parking duration. The assignment problem was solved by using a modified Frank-Wolfe algorithm, where the minimum path finding and assignment were done separately for each parking duration range, whereas the equilibrium and impedance calculations considered the whole population of parkers.

It should be noted that this problem is a special case of the problem of multimodal equilibrium assignment, which was studied extensively by Dafermos (7,8). In general, there are circumstances where the theoretical multimodal assignment problem has more than one solution. Even though it is impossible to prove the uniqueness of the solution, this problem does not appear to be significant within the existing framework.

There is a need to address the dynamic aspects of the problem. The assignment approach, by its very nature, deals only with steady-state analysis; that is, it is assumed that demands (and hence flow rates) are stable for a relatively long period, and the objective is to estimate those stable flows. In parking analysis, however, the major cause for capacity problems are not the flows of parkers in and out of lots, but rather the accumulation of parked cars. Moreover, to be able to approximate constant demands there is a need to analyze relatively short periods (morning peak, mid-day, and so forth). In such a case the number of parked cars in the start of the analysis period, and the change in that number during the period, are major determinants of the capacity of the system, and they had to be explicitly considered. It might have been possible to ignore this problem and to define some daily average volume-to-capacity ratios and resulting impedances, similar to the way 24-hr traffic assignments are done. However, this approach was rejected, mainly because of the apparent lack of sensitivity of the resulting model to relevant policy issues.

It is assumed that the rates of flow of vehicles into the system are constant during each analysis period. These include the flow rates of trips to every destination and to each parking lot, stratified by parking duration. With this assumption, the model is consistent. The assumption of fixed flow rates is somewhat inaccurate. For example, highly attractive lots are likely to be filled by early arrivals, whereas late arrivals might have to go to less-desirable places. However, this assumption does not significantly affect the major model outputs, which are an average impedance by destination and

parking duration and the degree of use of individual parking lots.

PARKING DISUTILITY

The disutility of parking at a given location included the following components:

1. Walking time--the time it takes to walk from the parking location to the destination by the shortest walk path;
2. Fee--the cost of parking at a given location;
3. Penalty cost--the expected penalty (fine) for parking illegally at the location;
4. Law-breaking cost--the tendency to avoid parking illegally, even if it is an attractive site from the point of view of the other components of disutility; and
5. Looking time--the time a person will have to wait for a parking space to be available at the location.

The form of the relationship used to calculate parking disutility is as follows:

$$DU(i,j,k) = Walk(i,j) * VW + Fee(i,k) * VF + Penalty(i,k) * VP + VB + Look(i) * VL \quad (1)$$

where

DU(i,j,k) = disutility of parking at location i for a parker with a destination j and a duration of k minutes,
Walk(i,j) = walk time from i to j,
VW = value of walking time,
Fee(i,k) = fee for parking at location i for duration k,
VF = weight placed on parking fees,
Penalty(i,k) = expected fine for parking at location i when the parking duration is k,
VP = weight placed on penalty costs,
VB = law-breaking costs,
Look(i) = time necessary to wait for an open parking space at parking location i, and
VL = value of looking time.

In application, weights placed on cost (VF and VP) are set equal to 1 while weights placed on time (VW and VL) have values related to the value of walking and looking time, and VB is given as a constant. The components of the disutility equation are determined as follows.

Walk Time

Walk time is determined from a walk network by finding the minimum walk time from each parking location to all possible destinations. If the walk links have adequate capacity, these values need only be computed once for a parking analysis.

Penalty Cost

The penalty from illegal parking is determined from the following relationship:

$$Penalty(i,k) = PrC(i,k) * Fine(i) \quad (2)$$

where PrC(i,k) is the probability that a parker of duration k will get caught parking illegally at location i, and Fine(i) is the fine for parking illegally at location i.

The probability of being caught is in turn related to the parking time limit and to police enforcement practices for checking violations. Two different cases exist. In the first case police must make one visit to determine if there is a violation (such as with parking meters, parking disks, and so forth). In such a case the probability of being caught is given by

$$PrC(i,k) = [Dur(k) - Limit(i)]/Int(i) \quad \text{with } 0 \geq PrC \geq 1 \quad (3)$$

where

Dur(k) = parking duration of parker k,
Limit(i) = time limit for parking at location i, and
Int(i) = enforcement interval; that is, the interval between police visits at location i to determine if someone is parked beyond the time limit.

Thus there is a 100 percent chance of being caught for illegal parking if the time limit is exceeded by an amount of time equal to the interval between police checks. If the limit is exceeded by half of the police interval there is a 50 percent chance of being caught, and so forth.

The second situation arises when it is necessary for the police to visit a location twice to determine if there is a violation (such as tire marks, recording of license-plate numbers, and so forth). Then the probability of being caught is given by:

$$PrC(i,k) = [Dur(k) - Int(i)]/Int(i) \quad \text{with } 0 \geq PrC \geq 1 \text{ and } Int(i) \leq Limit(i) \quad (4)$$

In this case there is a 100 percent chance of being caught if the parking duration is twice the enforcement interval. It is interesting to note that the chance of being caught in this case depends only on enforcement policies and is independent of the posted limit, assuming that the enforcement interval is greater than the posted limit.

Fee

The parking fee is simply the out-of-pocket cost of parking at a given location.

Looking Time

Looking time is defined as the average amount of time a person would have to wait at a given location for a parking space to be available. Unlike the previous components of utility, looking time will vary with the volume of parkers and the capacity of the given parking location. The relationship among looking time, lot size, and demand was determined as follows:

$$Look(i) = \{[AC(i) - 0.9 * NSP(i)]^2 * ADUR(i)\} \div [2 * NSP(i) * AC(i)] \quad \text{for } AC \leq 0.9 * NSP \quad (5)$$

where

Look(i) = average looking time for a space at location i,
AC(i) = maximum accumulation of parkers at location i,

NSP(i) = number of parking spaces at location i, and
ADUR(i) = average duration of parkers at location i.

This equation was derived by assuming that parkers would arrive uniformly over the analysis period for their given duration. The equation gives the average looking time for all parkers, including those that arrive early and do not have to wait. It was also assumed that parkers would have to begin to look for a space when the parking location was 90 percent full. The derivation of this relationship is given in the Appendix. Vehicle accumulations at a given location are calculated for each parking duration at each lot. Given the accumulations, look time can be determined and used in an iterative traffic assignment process to balance flows through the various parking lots.

TEST CASE

In order to observe the behavior of the model, a test network was prepared of the main shopping area in the city of Haifa, an area with severe parking problems in many sections. The area analyzed covers approximately 2 km² and contains about 600 legal parking spaces. The coded network is shown in Figure 3. The area was subdivided into 18 destination zones. The network contained 155 walk links and 32 parking lots.

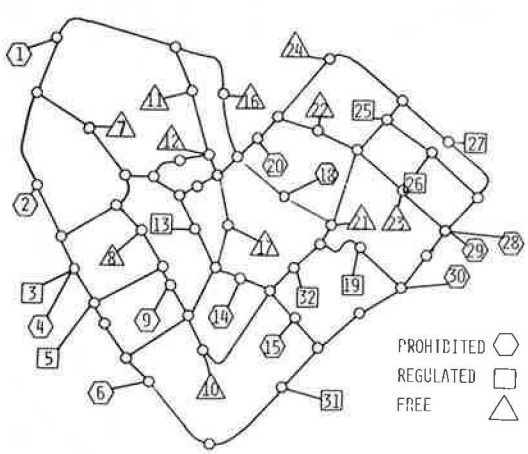

FIGURE 3 Test case network, western Hadar region, Haifa, Israel.

Actual experience in the area indicated a high degree of illegal parking, as well as overtime parking elsewhere. Thus no-parking and tow-away areas, as well as normal street parking locations, were included in the analysis. Most of the parking spaces in the area were either on-street or in open lots. Of the 600 legal parking spaces, about 400 were restricted to 1 hr. In addition, there were about 600 illegal parking spaces, about a third of which were in tow-away zones.

Parking links were taken as a side of a street and were characterized by the number of spaces, restriction type, parking fee, and police enforcement policy at that location. A single demand source linked to all parking locations was used as an initial loading point for all trips.

The model was exercised to determine the effects of various parameters and policies on model behavior. These tests included changes in the value of walking time, amounts of fines, police regulatory policy, and changes in parking supply. The results of these tests are discussed in the following sections. Three parking locations with different characteristics were chosen to illustrate model behavior. These are (a) lot 21, an outlying lot some distance away from the center of activity with unrestricted free parking; (b) lot 19, a close-in lot with a 60-min time limit and a parking meter rate of 5 shekels per hour (1 shekel = U.S. $0.02, as of July 1983); and (c) lot 18, a prime location with no parking at any time.

Initial parking fines for illegal parking were 100 shekels at lot 19 and 250 shekels at lot 18, the no-parking location. In each of the experiments the level of use of each lot will be given as an indicator of use. This is defined as the ratio of the number of parkers wishing to use the lot (accumulation) at the end of the analysis period to the number of spaces in the lot. The accumulations are equilibrium values after the iterative assignment process.

Changes in Value of Walk Time

In order to determine the effect of walking distance, the value of walk time was varied from 3 to 27 shekels per minute. The results of this analysis are shown in Figure 4. Considerable changes in parking use occur as the importance of walk time increases. Lot 21, where parking is free and unrestricted but located some distance away from the center, has a high level of use when the value of walk time is treated as being equal to looking time. This decreases as the importance of walk time increases. On the other hand, lots 18 and 19 show gains in use as walk time becomes more important. Both lots 18 and 19 carry penalties for overtime parking. At lot 19 there was a 60-min limit and a 100 percent chance of a fine after a 180-min duration. Detailed results indicate that short-duration parkers shift to this lot as the value of walk time increases. Lot 18 (where there is no parking, a more severe fine, and a 100 percent chance of being caught after 120 min) also has a concentration of short-duration parkers. Other parking lots that have towing restrictions and a no-parking time penalty at any time remain unused throughout the range of ra-

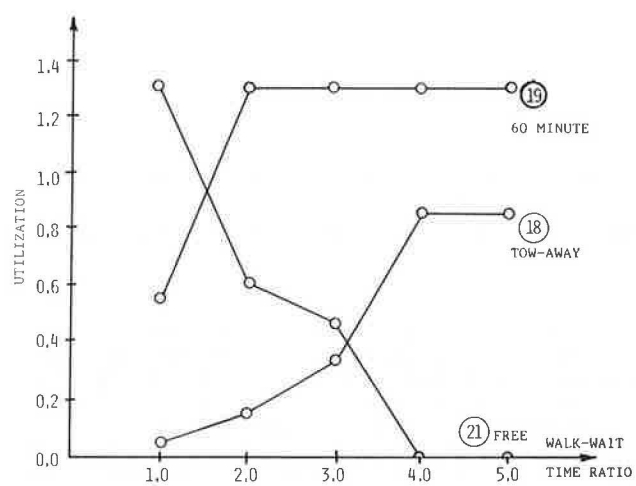

FIGURE 4 Parking location use as related to value of walk and wait time ratio.

tios used. Lots with convenient locations and no restrictions maintain a high level of use over the entire range of ratios of walk to look time.

Change in Fine Levels

A second parameter that was examined was the level of fines set for illegal parking. One component of the parking impedance at a given location is the expected penalty cost (fine) if illegal parking occurs. A series of runs were made with various fine levels, as shown in Figure 5. The base fine level of 1.0 represents fines of 100 shekels for exceeding a posted time limit, 250 shekels for parking in a no-parking zone, and 800 shekels for tow-away zone parking. Other runs were made with fines at 0.1, 0.5, 2.0, and 4.0 times the base level.

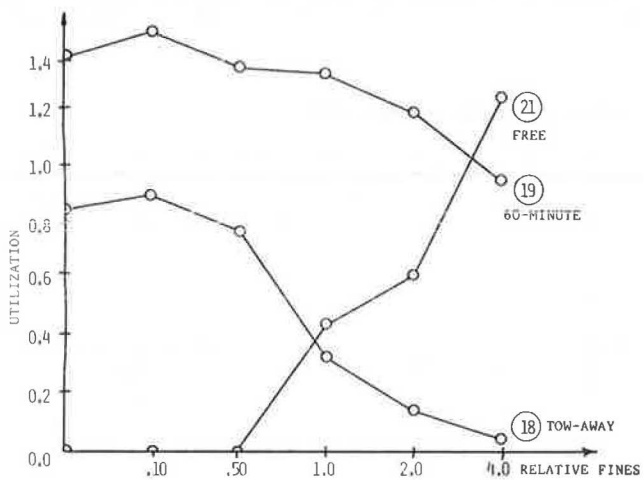

FIGURE 5 Parking location use as related to level of parking fines.

As can be seen in Figure 5, fine levels affect the distribution of parkers. By using the same lots as before, note that as the level of parking fines increases, there is a major shift away from lot 18 (no parking with the highest fine) and away from lot 19 (60-min parking) toward lot 21 where there is no time limit on parking. As would be expected, long-duration parkers, who are most likely to encounter a fine, shift away from the restricted lots.

Changes in Police Enforcement Interval

A second aspect of expected penalty cost relates to the enforcement policy of the police. This is represented in the model by the interval between checks by the police for illegal parking. As the police enforcement interval increases, the probability of getting caught for illegal parking will decrease, which has effects that are similar to a reduction in the level of fines. A series of runs were made with variations in the enforcement interval; these results are shown in Figure 6.

These results indicate a shift to illegal parking in more convenient locations (lots 18 and 19) and away from more distant, but unrestricted parking (lot 21). As the enforcement interval increases, the probability of being caught decreases for longer-term parkers; hence it is efficient for them to shift to more convenient locations, even though it may involve illegal parking. At lot 19 this attrac-

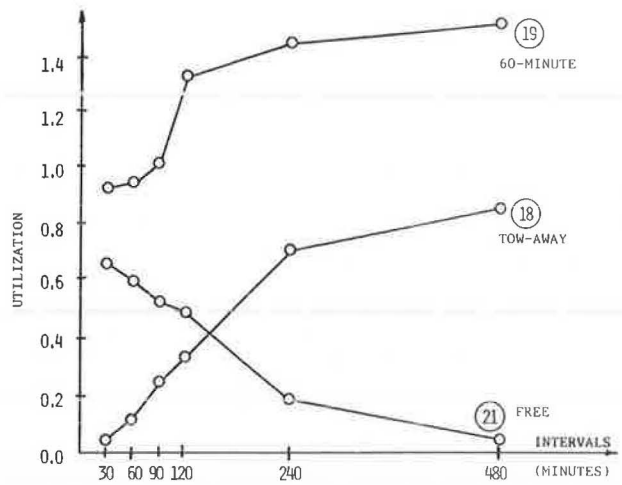

FIGURE 6 Parking location use as related to police enforcement interval.

tion is so strong that the lot is used more than the capacity; therefore, parkers have to wait for a space to be available.

Changes in Demand Level

Runs of the model were made with variation in the base demand to determine how the model would predict parking behavior under different demand levels. Those results are shown in Figures 7 and 8 and are

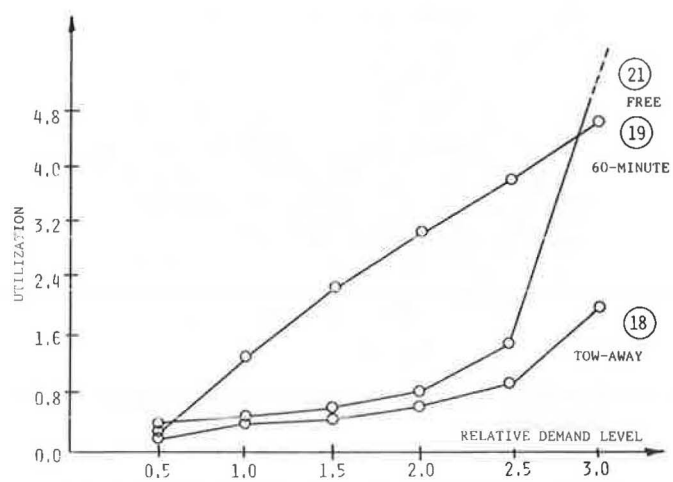

FIGURE 7 Parking location use as related to level of demand.

given in Table 1. As would be expected, use increases in the selected locations as demand increases. Lot 21 (where parking is unrestricted and free) has the most rapid increase, whereas lot 18 (a no-parking zone) has a slower increase. When demand reaches a level of approximately twice the base demand, all three locations are overloaded and have parkers waiting to use them.

When all parkers are compared by types of lots, as indicated by the data in Table 1, similar shifts can be observed. When demand is low, no vehicles use tow-away locations and only a small portion use no-parking zones. As demand increases, there is an overflow from unlimited and 1-hr locations to no-parking and tow-away zones. This occurs as the wait

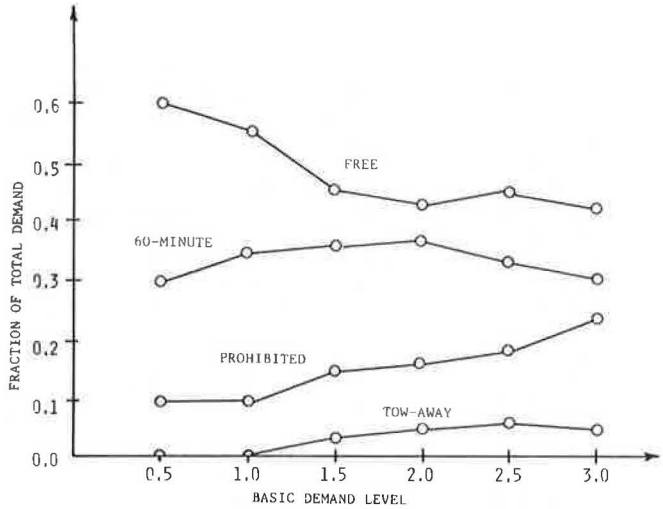

FIGURE 8 Split of parkers between lot types at different demand levels.

TABLE 1 Distribution of Parkers by Lot Type at Different Demand Levels

	No. of Parkers by Demand Level					
Type of Parking	0.5	1.0	1.5	2.0	2.5	3.0
Free	340	625	774	974	1,280	1,410
Regulated (60 min)	168	392	622	814	932	1,035
Prohibited	52	105	236	345	449	767
Prohibited (tow away)	0	0	49	108	142	151
Total	560	1,122	1,681	2,241	2,803	3,363

time at the unrestricted and 1-hr locations becomes so large as to make it attractive for short-term parkers to shift to no-parking zones, in spite of the increased risk of parking fines.

Additional Parking Facility

It was also possible to use the model to test the performance of a new parking facility at various locations in the network. For this analysis it was assumed that a 200-car parking facility would be added with a 5 shekel per hour fee for use. Furthermore, all street parking would be changed to a 5 shekel per hour fee parking with the same time restrictions as before. The results of these runs are given in the following table:

Location	Vehicles Parked	Maximum Accumulation
70	189	108
114	125	63
113	0	0

The use rate for the new facility is greatest at location 70. This location is the place with the highest use rate during the base case. The new facility at location 70 accommodates a total of 189 vehicles and has a maximum accumulation of 108 vehicles. When the lot is placed at location 114, which is about one block from location 70, demand drops to a total of 125 vehicles and an accumulation of 63 vehicles. At location 113, which is another block away, demand for the new facility drops to zero. In this situation there is sufficient capacity available closer to activity locations to accommodate all demand in the area without a new facility. Furthermore, if street fees are not raised to be comparable with the fees charged at the new facility, no demand is attracted to it. These comparisons relate to the base level demand, and use rates would vary with different demand levels. Such runs would be useful to set the size of the new facility, as well as to assess its economical potential.

These test runs demonstrate that it is possible to develop a model of parking behavior that is sensitive to a variety of pricing and policy options by use of a detailed description of parking disutility and equilibrium assignment procedures. Such an approach permits the analyst to test a variety of options; furthermore, it can demonstrate the sensitivity of parking patterns to varying factors.

CONCLUSIONS AND FURTHER DIRECTIONS

The parking model that is described in this paper appears to provide a sensible description of the parking process and to provide a method for estimating parking impedance that is compatible with other travel demand models. In addition, it also gives an estimate of the degree of use of different lots under different conditions. The model succeeds in responding in a consistent and logical manner to a wide range of policy changes, including the number of spaces available and their location, operating rules, changes in demand, enforcement intensity, and level of fines. The model also demonstrates the trade-offs among walking time, looking time, and the cost associated with parking. In particular, shifts are noted in parking patterns in response to changes in fines and enforcement practices.

The tests indicate that the model simulates reality satisfactorily; that is, it fills lots that are usually filled in reality, while leaving spaces in less desirable lots. It is, of course, desirable to fully calibrate the assumptions on which it is based to make it useful for policy analysis, even in its present form.

Much work still has to be done in devising and testing methods for relating policy factors to model variables. These include, for example, the translation of enforcement personnel to inspection intensity, and the representation of the efficiency of the court system (i.e., the probability of having to pay a fine when given a ticket). Some work also has to be done in finding methods to estimate the distribution of parking duration as a function of the type of activity in an area and the best way to subdivide the day into analysis periods. In addition, the alternative of conducting assignments by short time periods in a dynamic process should be explored.

APPENDIX: LOOKING TIME/VOLUME RELATIONSHIP

Given a parking lot with NSP spaces, an accumulation of AC, an average parking duration of ADUR, and uniform arrivals, the expected time for a parking space to be empty is given by the ratio of the parking duration to the number of spaces or

$$ADUR/NSP \qquad (A1)$$

If the location is full, the expected wait for the first extra vehicle is one-half of the expected time for a space to be available:

$$ADUR/(2 * NSP) \qquad (A2)$$

For the last extra vehicle, the waiting time is the time for the first phase wait time per vehicle times the number of extra vehicles:

$$[ADUR/(2 * NSP)] + \{[(AC - NSP - 1) * ADUR]/NSP\} \quad (A3)$$

Simplifying,

$$\{[AC - NSP - (1/2)] * ADUR\}/NSP \quad (A4)$$

The average wait for the excess vehicles is one-half of the wait for the last vehicle:

$$\{[AC - NSP - (1/2)] * ADUR\}/(2 * NSP) \quad (A5)$$

If the excess number of vehicles is large, the one-half can be dropped. Then the average wait for the excess vehicles is

$$[(AC - NSP) * ADUR]/(2 * NSP) \quad (A6)$$

The average wait for all parkers (i.e., looking time), including those who arrived before the lot filled, is

$$Look = \left(0 * NSP + \{[(AC - NSP) * (AC - NSP) * ADUR]/(2 * NSP)\}\right)/AC \quad (A7)$$

$$Look = [(AC - NSP)^2 * ADUR]/(2 * NSP * AC) \quad (A8)$$

If the wait time is expected to begin when the location is 90 percent full, then the looking time is

$$Look(i) = \{[AC(i) - 0.9 * NSP(i)] * ADUR\} \div [2 * NSP(i) * AC(i)] \quad (A9)$$

ACKNOWLEDGMENT

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Estimating Downtown Parking Demands: A Land Use Approach

HERBERT S. LEVINSON and CHARLES O. PRATT

ABSTRACT

A procedure for estimating and allocating downtown parking demand, based on land use and employment data, is presented. The procedure can be used to demonstrate how the peak parking accumulation can be prorated among analysis districts based on each district's share of employment (long-term parkers) and retail and service floor space (short-term parkers). The technique is similar to the gravity model approach to zonal interchanges. A case study of downtown New Haven, Connecticut, illustrates how the procedures are applied.

The traditional approach to estimating downtown parking demands involves detailed surveys of parker characteristics. Parkers along curbs and in lots and garages are queried as to their times of entry and exit, trip purpose, downtown destinations, and fee paid. The parking demands for each downtown block or analysis area are then obtained by aggregating parker destinations for each hour of the day. The peak demands for each analysis area are compared with the available spaces to identify parking space surpluses and deficiencies.

This procedure has been widely used for more than 25 years in parking planning and feasibility studies. Yet it is time consuming and costly because detailed surveys are required on a block-by-block basis. Sampling procedures have limited application because of the strong ties between where motorists park and where they are destined.

The need for a simple, yet reliable means of obtaining data on downtown parking demands is addressed. In this paper it is demonstrated how land use and employment data can be used to estimate demands on a subarea basis, and the concept is illustrated with a parking demand study for downtown New Haven, Connecticut.

CONCEPT

The concept is relatively straightforward. It is similar to that used by the gravity model to estimate zonal interchange.

1. The peak accumulation of parkers, as obtained for parking use studies, is assumed to approximate the aggregated hour-by-hour downtown parking demands.
2. This demand is allocated to the various subareas based on each area's relative share of downtown activity.

The data requirements include

1. Measures of the hour-by-hour accumulations of parkers within the downtown area,
2. Estimates of downtown floor space and employment by analysis district and type of use, and
3. Estimates of the approximate proportion of the total parking accumulation that represents long-term parkers.

Where information is available on the distribution of the peak parking accumulation between long-term (more than 3 hr) and short-term (less than 3 hr) parking, downtown employment can be used to allocate long-term parkers, and retail and service floor space can be used to allocate short-term parkers. Where this distribution is not available, either total employment or total nonresidential floor space can be used.

The following formula summarizes the procedure:

$$d_i = A_L \cdot (e_i/\Sigma^j e) + A_S \cdot (F_i/\Sigma^j F) \qquad (1)$$

where

d_i = peak parking demand for zone i,
A_L = total peak long-term accumulation of parkers,
A_S = total peak short-term accumulation of parkers,
e_i = employment in zone i,
e = total employment in central business district (CBD),
F_i = retail and service floor space in zone i,
F = total retail and service floor space, and
j = number of zones.

A simplified three-zone example (Table 1) illustrates the procedure. The peak accumulation of 4,000 parkers includes 3,000 long-term parkers and 1,000 short-term parkers. The 3,000 long-term parkers are allocated to each zone based on its share of the total employment--60 percent in zone 1, 20 percent in zone 2, and 20 percent in zone 3. The 1,000 short-term parkers are allocated based on each zone's share of the retail and service floor space--33 percent in zone 1, 50 percent in zone 2, and 17 percent in zone 3. The long- and short-term demands for each zone are then added to obtain each zone's total demand--2,130 spaces in zone 1, 1,100 spaces in zone 2, and 770 spaces in zone 3.

CASE STUDY

The procedure was applied to downtown New Haven as a part of an overall traffic and parking study. The parking analysis region lies to the north of the Route-34 Expressway and contains major shops, offices, government buildings, and the main Yale University campus. The employment in the area exceeds 20,000, and about 80 percent of all peak-hour trips to this area are made by car. Parking facilities are provided outside of the area to accommodate demands in peripheral places. For example, a 2,400-space garage serves the Yale-New Haven Medical Center.

The parking analysis area (Figure 1) contains about 13,680 spaces. Field surveys found a peak accumulation of 9,770 parkers. Records of the New Haven Parking Authority garages found that more than 70 percent of the maximum accumulation remained for more than 3 hr. (The percentage for downtown Boston

TABLE 1 Illustrative Example

	CBD Activity				Parking Demand				
	Employment		Retail and Service Area		Long-Term Parkers		Short-Term Parkers		
Zone	No.	Percent	Floor Space (ft^2)	Percent	Percent	No.	Percent	No.	Total
1	3,000	60	200,000	33	60	1,800	33	330	2,130
2	1,000	20	300,000	50	20	600	50	500	1,100
3	1,000	20	100,000	17	20	600	17	170	770
Total	5,000		600,000			3,000		1,000	4,000

Note: Peak accumulation = 4,000; long term accumulation = 75 percent (or 3,000); and short term accumulation = 25 percent (or 1,000).

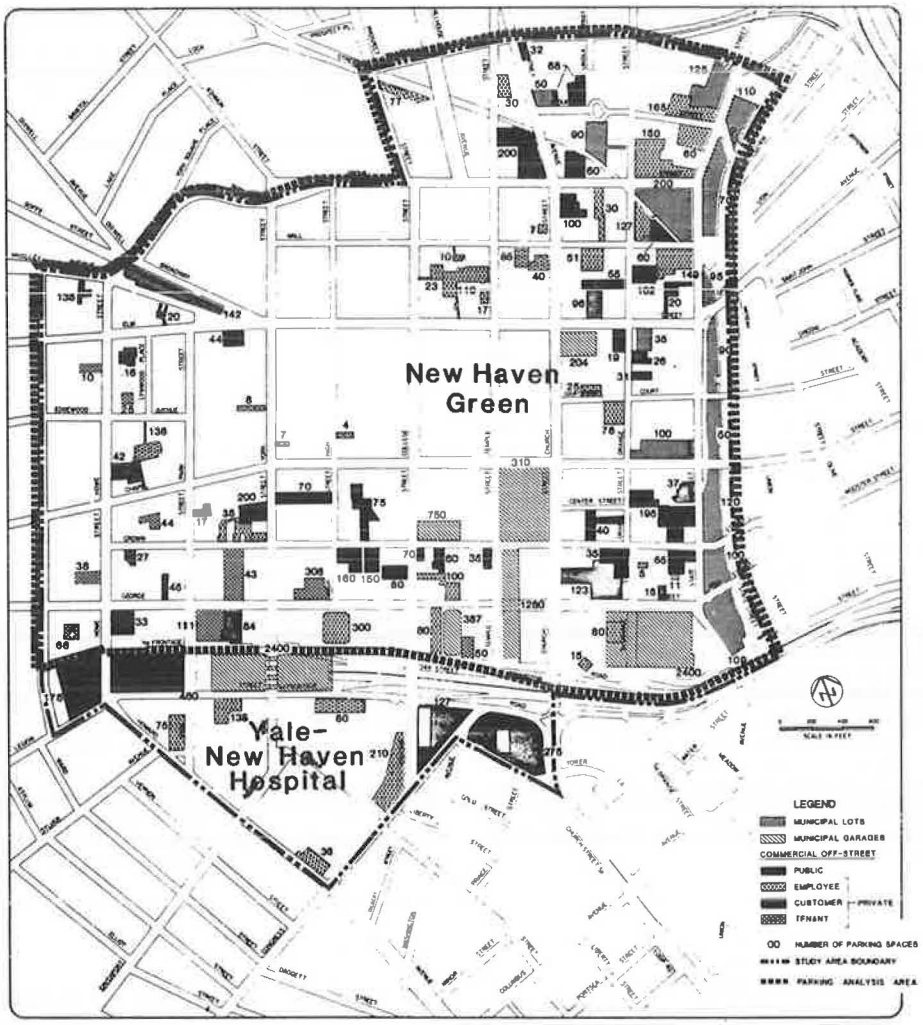

FIGURE 1 Off-street parking space, downtown New Haven, 1982.

was 79 percent.) Figure 2 shows the hour-by-hour accumulation for long- and short-term parkers.

The following steps were applied in estimating parking space demands and needs for each of the seven analysis districts located to the north of the Route-34 Expressway.

1. The parking supply within the analysis area was approximated to be 13,700 spaces. The effective supply, based on an efficiency factor of 0.9, was approximated to be 12,300 spaces. This reduction accounts for the fact that all facilities cannot be 100 percent occupied at the same time.

2. The maximum accumulation of parked vehicles was approximated to be 9,800 spaces. This number was increased slightly to 10,000 to allow for "walk-in" parkers. The long-term accumulation (more than 3 hr) was assumed to be 7,000, and the short-term accumulation 3,000. These figures were based on the 70-30 split between the long- and short-term accumulation in New Haven Parking Authority garages. (Earlier analysis revealed the numbers to be 6,900 and 2,870 based on the 9,770 spaces.) These numbers were assumed to represent the long- and short-term demand within the area.

3. Floor space employment estimates were ob-

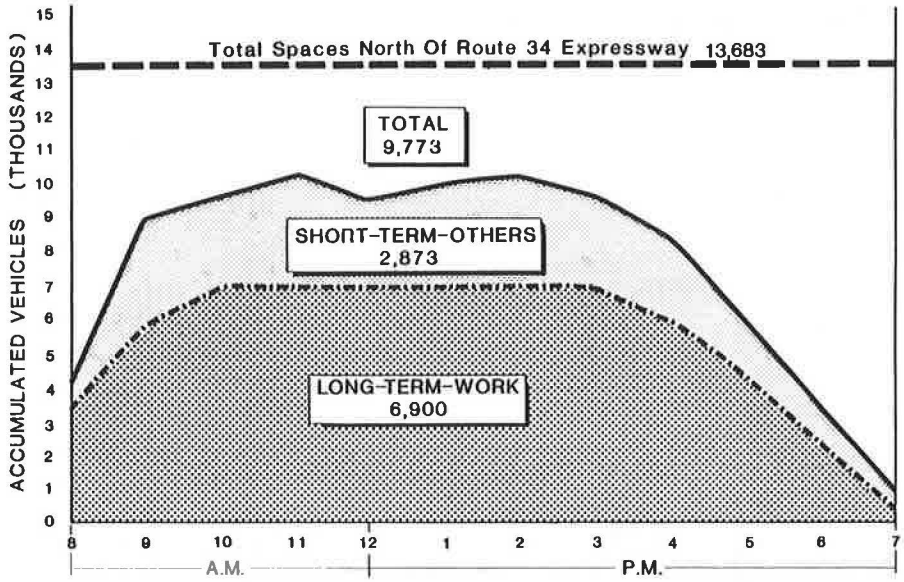

FIGURE 2 Parking accumulation north of Route-34 Expressway by trip purpose, typical weekday, 1982 (downtown New Haven).

tained from the city of New Haven. These estimates, although subject to variation, indicate the relative intensity of activity within each analysis district. It was reasoned that the parking demands in each district would reflect the relative amount of total activity in that district. (Reported employment figures tend to overstate the actual daytime work force because they do not account for absentees and vacations, people who do not work 5 full days, and workers who travel.)

4. Accordingly, 6,500 spaces of the long-term demand of 7,000 spaces were allocated to each analysis district based on its share of the existing downtown employment. The remaining 500 spaces represent Yale-New Haven Hospital parkers in the Coliseum Garage, who were allocated to district 3. (The hospital complex is located south of the Route-34 Expressway, and the parkers were bused to the garage in 1982 before the new 2,400-space Air Rights Garage opened.)

5. The 3,000 spaces of short-term demand were allocated to each analysis district based on its share of the existing total retail and service floor space.

6. The total demand for each district was obtained by adding the short- and long-term demand (steps 4 and 5).

7. The total parking demand was compared with the effective parking supply to obtain an estimate of existing space surpluses and deficiencies (i.e., needs).

The results of this analysis are given in Tables 2 and 3 and are shown in Figure 3. The data in Table 2 give the demand calculations; the data in Table 3 give parking space supply, demands, and needs by analysis district; and Figure 3 graphically summarizes parking space surpluses and deficiencies. Overall, there is a surplus of some 2,315 spaces in downtown New Haven. A large surplus--979 spaces--exists in district 1, located in the Arts Center area. Many parking spaces provided in this area serve the government and financial center. Substantial space surpluses are also found in districts 3, 4, and 6. Districts 7 and 8, largely occupied by Yale University, are currently in balance. The analysis indicates a deficiency of more than 300 spaces in the government and financial district (district 2).

It is interesting to note that the peak long-term demand of 6,500 spaces approximated 0.32 spaces per reported downtown employee. Similarly, the short-term demand approximated 1.46 spaces per 1,000 ft^2 of retail and service floor space.

TABLE 2 Estimated 1982 Peak Parking Space Demand by Analysis District (New Haven CBD)

	Long-Term Parking Demand			Short-Term Parking Demand			
District	Employees	Percent of Total	Spaces	Retail Plus Service Floor Space[a]	Percent of Total	Spaces	Total Parking Demand Spaces
1	2,155	10.7	696	97,538	4.7	141	837
2	3,926	19.4	1,261	223,734	10.9	327	1,588
3	6,340	31.3	2,535[b]	1,032,372	50.2	1,506	4,041
4	4,669	23.1	1,501	300,110	14.6	438	1,939
6	413	2.0	130	222,600	10.8	324	454
7	739	3.7	240	170,650	8.3	249	489
8	1,991	9.8	637	10,000[c]	0.5	15	652
Total	20,233		7,000	2,057,004		3,000	10,000[d]

Note: Table is based on data obtained from J. Farnham, city of New Haven.

[a] Excludes Malley's (in square feet).
[b] Includes 500 Yale-New Haven Hospital parkers.
[c] Assumed for allocation purposes.
[d] 9,773 peak accumulation rounded up to 10,000 to reflect walk-in traffic.

TABLE 3 Estimated Parking Space Surpluses and Deficiencies, Downtown New Haven, 1982

District	Supply			Effective Supply[a]	Demand			Surplus	Deficiency
	Curb	Off-Street	Total		Long Term (work)	Short Term (other)	Total		
1	214	1,804	2,018	1,816	696	141	837	979	
2	65	1,349	1,414	1,273	1,261	327	1,588		315
3	84	4,940	5,024	4,522	2,535	1,506	4,041	481	
4	97	2,605	2,702	2,432	1,501	438	1,939	493	
6	250	921	1,171	1,054	130	324	454	600	
7	205	403	608	547	240	249	489	58	
8	489	257	746	671	637	15	652	19	
Total	1,404	12,279	13,683	12,315	7,000	3,000	10,000	2,630[b]	315

[a] Space use efficiency factor of 0.90.
[b] Net surplus = 2,315.

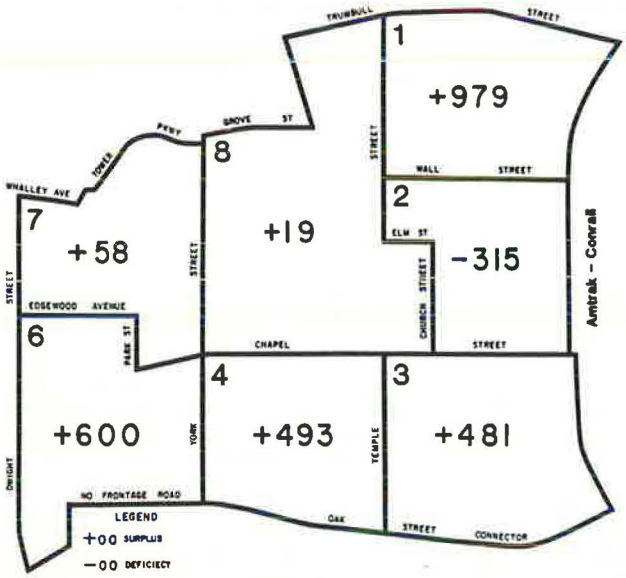

FIGURE 3 Estimated parking space needs, 1982.

PLANNING IMPLICATIONS

The use of employment and floor space data to allocate downtown parking demands provides a cost-effective approach to parking demand estimation. It also produces reasonable parking indices for use in estimating the impact of new downtown development. Its data needs, like the analysis steps, are simple.

The procedures need estimates of floor space and employment by type of use and the peak daytime downtown parking accumulation. The long- and short-term components of demand can be estimated from parking garage records or from the parameters identified in this paper (70 to 80 percent long term). Alternatively, the components can be estimated by using the 9:30 a.m. accumulation as the long-term estimate; the difference between that value and the maximum accumulation can be used at the short-term demand. Ideally, information should be obtained on the accumulations of people who park outside of the study area and have destinations within it to reflect unsatisfied or latent demand.

The demand analyses are based on the relative intensity of activity within each district. Thus they do not consider the unique characteristics of specific land uses that may raise or lower demands, especially in individual blocks. Some service and retail activities attract 5 to 10 times as many daily short-term parkers per 1,000 ft² as others (e.g., a doctor's office as compared with a piano store). Where such areas of high generation are significant in magnitude, it may be desirable to weight their floor areas occasionally. However, the differences in peak accumulation are usually less than those relating to daily parking demand. Moreover, nonwork demands represent the smallest component of total parking demand, and many visits to high-demand activities are made by downtown workers. Therefore, such adjustments are not essential for an overall CBD parking analysis.

The procedure works best in city centers where three-quarters or more of the downtown work force arrives by car and the differential impacts of public transport on CBD parking demand are not significant. In this case it is reasonable to assume that the proportions of the downtown daytime population arriving by transit are relatively uniform among analysis areas. Where there is heavy transit use, especially in city centers served by rail transit, allowance must be made for proximity to major rail stations. Consequently, the procedure is more applicable in New Haven rather than in New York, and in Providence rather than in Philadelphia.

Several adjustments may be necessary to compensate for CBD parkers who use outlying park-and-ride lots or people parking in the city center that have destinations outside of it.

1. Park-and-ride areas outside of the city center represent an additional component of downtown demand. However, they do not significantly affect the allocation of parking demands within the downtown corridor. They should, however, be included in the total CBD demand, and should be allocated on the same basis as work trips.
2. Where CBD parking facilities serve areas located outside of the city center, these outlying areas should be incorporated as additional analysis districts.

Accuracy of floor space and employment poses another constraint.

Despite these qualifications and constraints, the procedure can produce a realistic picture of current parking demands in the center of most cities in a cost-effective manner. It also provides a basis for estimating the effects of additional downtown development. Additional research is, of course, desirable to compare the results of this method with demands derived from interviews of parkers.

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Effects of Ending Employer-Paid Parking for Solo Drivers

MONICA SURBER, DONALD SHOUP, and MARTIN WACHS

ABSTRACT

The change in employee travel choices at a company in Los Angeles that ended employer-paid parking for solo drivers who do not use their cars at work is documented. The modal split among affected employees changed in the following ways: solo driving fell from 42 to 8 percent, carpooling rose from 17 to 58 percent; and bus ridership declined from 38 to 28 percent. There was no change in the modal split at a nearby comparison company that continued to offer free parking to all employees. It is concluded that ending employer-paid parking for solo drivers significantly influenced employees' modal choices.

Commuter Transportation Services, known popularly as Commuter Computer, was founded in 1974 as a private nonprofit corporation to promote ridesharing in southern California, and since that time its transportation subsidy policy has evolved toward consistency with its mission. In 1974 all employees were offered free parking as a fringe benefit. In 1976 each vanpooler was offered a subsidy equal to the price of a parking space. In 1979 bus riders were offered free transit passes. And in 1981 there was a decision at Commuter Computer to phase out parking subsidies for the 70 percent of employees who did not use their cars for work. Carpoolers continued to park free, and the transit pass program was unaltered.

Commuter Computer is located on Wilshire Boulevard, a central transit corridor near the Los Angeles central business district (CBD). Until May 1982 Commuter Computer paid $57.50 a month per space to rent parking spaces that it offered free to its employees, so the parking subsidy for each solo driver was $57.50 a month. This subsidy was eliminated in two phases. Beginning in May 1982, the parking subsidy for solo drivers was reduced to $28.75 a month. Those people who continued to park in the building paid $28.75 per month for a space that cost Commuter Computer $57.50 a month, and those who continued to drive alone and park elsewhere were reimbursed for half their cost of parking, up to $28.75 per month. In May 1983 the parking subsidy for solo drivers who did not use their cars for work was ended. Solo drivers then paid $57.50 a month to park in the building or chose from their other options, which included some lower-cost parking lots and scarce on-street parking in a nearby residential neighborhood.

EFFECTS OF ENDING PARKING SUBSIDIES FOR SOLO DRIVERS

The program at Commuter Computer was examined to discover the effects of eliminating free parking for solo drivers. Accounting records supplemented by telephone interviews of employees provided data on travel mode for all affected employees from January 1982 to July 1983. The data in Table 1 and in Figure 1 present the results for the 70 percent of employees who did not use their cars for work. The 30 percent who used their automobiles at work were omitted from the analysis. It is clear that there was a sudden reduction in solo driving immediately following each of the two reductions in parking subsidy.

Solo driving fell from an average 42 percent during the last 4 months when solo drivers parked free to 9 percent during the first 3 months when they

TABLE 1 Modal Choice of Employees

Date	Affected Employees[a]	Modal Choice of Employees (%)				Employee Parking ($/month)	Parking Subsidy ($/month)
		Solo	Carpool[b]	Bus	Other		
1982							
January	62	39	19	42	0	0	57.50
February	68	40	16	40	4	0	57.50
March	69	40	16	38	6	0	57.50
April	73	48	18	30	4	0	57.50
May[c]	72	33	32	30	5	28.75	28.75
June	72	37	30	33	0	28.75	28.75
July[d]	71	35	30	35	0	28.75	28.75
August	70	30	36	34	0	28.75	28.75
September	68	22	41	32	4	28.75	28.75
October	64	25	41	34	0	28.75	28.75
November	65	22	41	37	0	28.75	28.75
December	63	21	43	36	0	28.75	28.75
1983							
January	65	24	38	38	0	28.75	28.75
February	67	24	42	33	1	28.75	28.75
March	65	25	43	31	1	28.75	28.75
April	60	21	47	30	2	28.75	28.75
May[e]	61	8	61	28	3	57.50	0
June	57	7	60	26	7	57.50	0
July	55	9	54	29	7	57.50	0

Note: From January to April 1982 there was full parking subsidy; from May 1982 to April 1982 there was half parking subsidy; and from May to July 1983 there was no parking subsidy.

[a] Excludes the 30 percent of employees who continued to receive free parking because they use their cars for work.
[b] Only two employees are in a vanpool; thus they are included in the analysis as carpoolers.
[c] Parking subsidy for solo drivers was reduced to $28.75.
[d] Proposition A reduced regular bus pass price from $34 to $20; permit price was raised to $5.00.
[e] Parking subsidy for solo drivers ended.

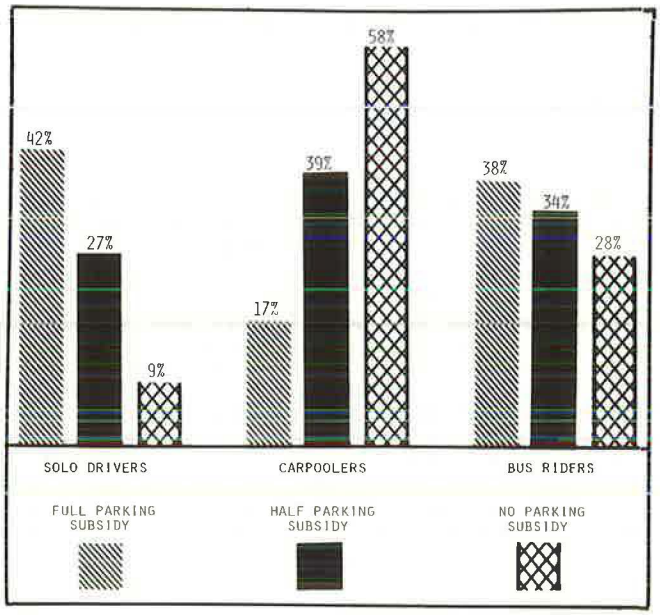

FIGURE 1 Employee modal choice and parking subsidy policy.

paid the market price to park. Of the five remaining solo drivers, as of July 1983, only one was willing to pay the $57.50 a month to park in the building. The other solo drivers parked in a cheaper ($20.00 per month) lot one block away. The share of employees carpooling or vanpooling rose from an average of 17 percent to 58 percent, and the proportion riding the bus fell from 38 to 28 percent. A Z test for the significance of proportional changes between the two periods indicated that the number of solo drivers and the number of carpoolers was significantly different from what would be expected by chance, but the decrease in transit use was not statistically significant.

From the pattern of transit change it appears that many solo drivers invited bus riders to join them as carpoolers because it saved the solo driver $57.50 a month plus it split the driving cost. Second, the cash value of the carpooling subsidy was greater than that of a transit pass. A regular transit pass price is $20.00, whereas the cash value of a parking permit for two persons carpooling is $28.75 each. Thus employees were subsidized more to carpool.

The data in Table 2 give a rough estimate of what desubsidizing solo drivers did to average vehicle occupancy. It is assumed that (a) all carpools consist of two persons, both before and after desubsidizing solo drivers, and (b) 66 employees were affected (the average number of affected employees over the 19 months). Given these assumptions, the data in Table 2 indicate that the average vehicle occupancy rate rose from 1.2 to 1.8 people per car. Although 23 fewer employees drove to work alone, 26 more employees carpooled to work, so the net result is that 3 more people drove to work in 10 fewer cars. Eight percent more employees came to work by car, but the number of cars driven to work fell by 29 percent.

To test whether employment turnover affected the results, records of modal choice were reviewed for a subsample of persons employed both before and after the first reduction phase. The change in modal choice for this smaller sample of 50 employees was quite similar to the pattern for all employees included in the previous analysis. This finding strengthens the argument that price, and not some other factor such as employment turnover, explains the modal shift.

CONTROL COMPANY

As part of its rideshare matching service to client companies, Commuter Computer collects and analyzes companies' modal-split statistics. The modal-split data from a nearby company similar to Commuter Computer were used as a control. The data in Table 3 show the similarity of Commuter Computer and the control company.

Figure 2 shows the modal split at each company for April and December 1982, the two most recent dates for which data were available from the control company. During this time period the first phase of the subsidy reduction at Commuter Computer was initiated. Solo driving declined by more than half at Commuter Computer and carpooling more than doubled, whereas solo driving rose slightly and carpooling remained constant at the control company. Bus ridership increased at Commuter Computer and decreased slightly at the control company.

The comparison of Commuter Computer with the control company leads to the conclusion that the reduction in subsidy to solo driving, and not some unknown exogenous factor, is the likely cause of the changes in commuting behavior at Commuter Computer.

FINANCIAL IMPACT

Commuter Computer's cost of providing commuter allowances to employees declined 15 percent from January 1982 to July 1983, during a period when the price per space to the company increased to $60.00 per month. At the same time the cost per bus pass dropped in July 1982 from $34.00 to $20.00 because of a new sales tax enacted in the county that was tied to a general reduction in bus fares.

This modest saving was, in Commuter Computer's case, essentially a bonus because desubsidizing solo driving was based on principle and was not done primarily for financial reasons. Had parking subsidies also been discontinued for carpoolers, the outcome would have been different. Of the more than $3,000 spent on commuter allowances in July 1983, 34 percent was for carpools. Subsidies to bus riders, in contrast, constituted only 12 percent of the July commuter allowance as a result of both lower unit cost and lower use. Two carpoolers now get a subsidy

TABLE 2 Effect of Parking Subsidization on Vehicle Occupancy Rates

Subsidy	Solo Drivers (%)	Carpoolers (%)	No. of Solo Cars	No. of Carpool Cars	Total Cars	People in Solo Cars	People in Carpools	People per Car
Full	42	17	28	6	34	28	12	1.2
Half	27	39	18	13	31	18	26	1.4
No	8	58	5	19	24	5	38	1.8

Note: It is assumed that 66 employees were affected by desubsidization, and that all carpools consisted of two persons.

TABLE 3 Commuter Computer and Control Company Comparison

	Commuter Computer	Control
Location	Wilshire Corridor, 3300 block	Wilshire Corridor, 3400 block
Transit	Five bus lines directly pass building	Six bus lines directly pass building
Size	Approximately 100 employees	Approximately 100 employees
Job-related automobile use	Approximately 30 percent	Approximately 5 percent
Building parking price	$57.50 per month per space	$42.50 per month per space
Transportation fringe benefits		
Solo drivers	Free parking until May 1982, then $28.75 until May 1983, then zero	Free parking
Vanpools	$57.50 per month per vanpool	Free parking
Carpools	Free parking	Free parking
Transit	Free bus pass	Free bus pass

of $30.00 per month each, whereas a bus rider gets a subsidy of only $20.00 per month. The continued subsidization of parking helps to explain the decline in bus ridership. If all parking subsidies had been entirely withdrawn, bus ridership might have increased.

COMPARISON TO OTHER STUDIES

The results at Commuter Computer are consistent with those found in a number of other studies documenting the effects of a change in the price of parking on commuter modal split. A brief summary of these studies is given in Table 4. Only three studies document the results of reductions of parking subsidies by employers. Two other studies compare two similar groups in which one group's parking is subsidized by the employer and the other group's is not. Two studies present results of reducing rates for carpoolers, and the remaining ones deal with price increases in the form of time-specific surcharges or a tax. For a fuller description of these studies, see Miller and Higgins (1).

These studies vary widely with respect to both the effects of price and initial conditions, such as the extent of parking supply involved, the availability and price of alternative parking options, the availability and quality of transit service, ride-sharing opportunities, and the incentives offered to use a particular mode. Depending on these and other factors, a change in the price of parking can dramatically change the modal split, as evidenced by the change at Commuter Computer, or have no effect, as was the case in Madison.

DISCUSSION OF RESULTS

When free parking was offered to all employees at Commuter Computer, carpoolers saved nothing on parking. Now only carpoolers park free, and solo drivers who joined a carpool each saved $57.50 a month on parking. Thus it is not surprising that ending employer-paid parking for solo drivers sharply increased carpooling.

Another way to show why ending free parking for solo drivers so strongly influenced modal split is to estimate its impact on the total out-of-pocket cost of driving to work. The average round trip to and from work in southern California is 20 miles; if the national fuel economy average of 20 miles/gal is assumed, the average work trip uses 1 gal of gasoline a day. At $1.25/gal and 22 working days per month, gasoline for the average commuter costs $27.50 a month. Therefore, ending the $57.50 per month parking subsidy for solo drivers raised the solo driver's out-of-pocket cost of gasoline and parking from $27.50 a month to $85.00 a month. This cost increase for the average 20-mile trip is equivalent to an increase in the cost of gasoline from $1.25 to $3.86/gal.

An alternative approach that could have been taken by the company would have been to offer all employees a cash travel allowance rather than subsidized parking (2,3). Employees would then have a choice of paying for their own parking or choosing another mode, with the option of pocketing the difference if a less-expensive alternative were chosen. A discouragement to this alternative is that employees are not subject to tax for the cash value of

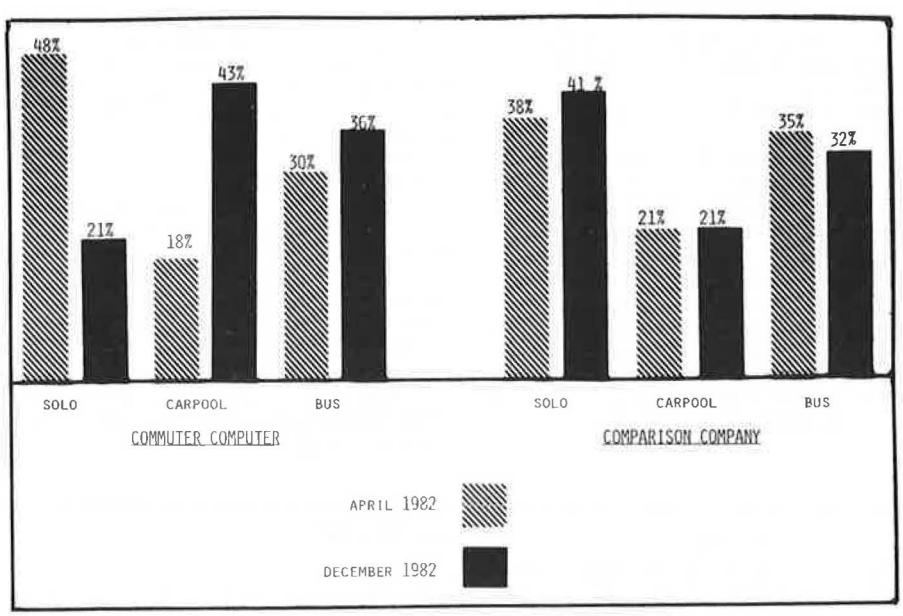

FIGURE 2 Modal split of Commuter Computer and comparison company.

TABLE 4 Parking Price Impacts on Modal Use

Study Location	Price[a]	Modal Split	Other Conditions
Reduced Employee Subsidies			
Bellevue, Washington CBD—1982 *(1)*	Pre-1982 employees provided free parking, poolers also given $35; post-1982 solo driver employees pay $35 to park, poolers park free, other modes paid $10	36 percent of all employees nonsolo mode, 23 percent carpool	No on-street free parking, little commercial parking
District of Columbia, city and suburban—1980 *(1)*	$0-$33 at all government lots in metropolitan area	1-10 percent automobile use reduction in city; 2-4 percent drop in suburban sites	Free on-street parking in some areas; transit level varied
Ottawa, Ontario, Canada CBD—1975 *(1)*	$20-$24 increase to 70 percent of commercial rate at all federal spaces	20 percent drop in solo automobile use, 16 percent rise in bus use	High level transit, limited parking
Subsidized and Nonsubsidized Comparisons			
Century City, California (high density employment center)—1976 *(2)*	Pays $40 a month for parking Pays approximately $20 a month Pays $0 per month	75 percent solo, 13 percent pool 85 percent solo, 9 percent pool 92 percent solo, 4 percent pool	Limited parking, high congestion, medium-high level transit
Los Angeles—1961 *(2)*	Pays $16 a month for parking Pays $0 a month for parking	40 percent solo, 27 percent pool, 3 percent bus 72 percent solo, 16 percent pool, 12 percent bus	Limited parking, high congestion, high level transit
Reduced Rates for Carpoolers			
San Francisco, near CBD—1980 *(1)*	$35-$60 reduced to $10 at three state lots	Attracted poolers from other lots (85-90 percent), from transit (3-5 percent), from solo (3-5 percent)	High level transit
Seattle, near CBD—1974 *(1)*	$25 permit reduced to $0 and $5 at two city lots	Attracted poolers from other lots (38 percent), from transit (40 percent), from solo (22 percent)	High level transit
Other Parking Price Change Studies			
Madison, Wisconsin (high density state capital and university)—1981 *(1)*	$1.25 surcharge at three off-street facilities between 6:30-9:30 a.m.	No shift to carpools or transit, shifted to other facilities	High level transit
Eugene, Oregon (city core)—1980 *(1)*	$16 increase at two garages; $6-$16 increased to $16-$24 at several lots	200 fewer permit sales; 40-50 carpooling, 30-40 used shuttle	Medium level transit; carpools (3 persons) park free; carpools (2 persons) get 20 percent off; free parking and shuttle from outlying lot
Chicago CBD—1978 *(1)*	30-120 percent increase at eight city lots	Aggregate 35 percent fewer cars, shorter duration, 72 percent decline in pre-9:30 a.m. parkers	Transit predominant CBD mode, short-term rates lower than commercial rates
San Francisco—1970 *(1)*	25 percent tax on off-street parking at 13 city garages	No. of parked cars declined at seven lots, increased at six lots, duration declined	High level transit, variation in competing lots

[a]Price column shows different values for each category, as follows: reduced employee subsidies = price increase; subsidized and nonsubsidized comparisons = price differences; reduced rates for carpoolers = price reduction; and other parking price change subsidies = price change.

parking supplied as a fringe benefit. Adding a travel allowance to taxable income would be opposed for this reason. Although federal legislation has been discussed to make this allowance tax free, no action has been taken to date.

Because the employees in this study worked for a ridesharing agency, it could be assumed that they were more likely to rideshare than employees whose business was not the promotion of ridesharing. But it could also be argued that because they work at Commuter Computer and are already aware of all the benefits of ridesharing, those who continue to drive alone would be a group less prone to rideshare than a similar group of solo drivers not already aware of the benefits. In any case, the economic incentive for switching modes was undoubtedly more critical than the nature of the business of the firm.

CONCLUSION

Ending free parking for solo drivers at Commuter Computer dramatically reduced solo driving. Solo driving decreased from 42 percent of the modal split during the last 4 months of free parking to 8 percent during the first 3 months after the parking subsidy for solo drivers was ended. Carpooling rose from 17 to 58 percent, and bus ridership declined from 38 to 28 percent during the same period. Vehicle occupancy among those driving to work is estimated to have risen from 1.2 to 1.8 persons per car.

The situation at Commuter Computer was unique in several respects: the parking subsidy was removed only for solo drivers who did not use their cars for work; and carpools, vanpools, and bus riders continued to receive subsidies. The organization's mission is to promote ridesharing and to provide rideshare matching services, and matching services were immediately available to all employees.

Given these qualifications, this case study demonstrates that employer-paid parking for solo drivers encourages solo driving, and that ending employer-paid parking for solo driving can greatly encourage ridesharing.

ACKNOWLEDGMENT

The authors would like to thank Peter Valk, Director of Planning and Development at Commuter Computer, for his generous help.

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Effects of Parking Measures in the Center of Leeuwarden

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ABSTRACT

In the center of Leeuwarden, a town with approximately 85,000 inhabitants in the northern part of The Netherlands, new parking regulations were introduced that caused radical changes in parking policies. A before-and-after study has been carried out to get information about the effects of the new parking policy. The effects can be divided into the effects on the parking system (primary level), the effects in relation to the transport system (secondary level), and the effects on the spatial and economic system (tertiary level). The situation before the introduction of the parking measures is compared with the situation a year after their introduction. The results of the before-and-after study are discussed in detail.

Parking in the town centers in The Netherlands is a matter of constant concern. Various categories of motorists require parking space in the town center, but people who work there often occupy much of the parking space. Consequently, the number of parking places remaining for residents, tradespeople, and shoppers and visitors to businesses are believed to be insufficient. Ease of parking for people visiting town centers thus leaves much to be desired.

A large number of municipalities have therefore started regulating the use of parking space by residents, persons working in the city centers, and visitors. The principal measures used are those that restrict parking duration or that owe their effect to the operation of a price mechanism.

It is important to understand the effects of these measures in the urban centers. Do they serve their purpose? Are there any unexpected side effects? What effect do they have on the parking behavior of people working in and visiting the center?

The Project Bureau for Integrated Transport Studies had the opportunity to answer some questions of this nature, in consultation with the Leeuwarden municipal authorities. In November 1979 parking measures were introduced in the Leeuwarden city center that altered the parking situation drastically. Several surveys were carried out to determine the effects of these measures.

The results of the parking surveys are reported. The parking situation before the introduction of the measures is reviewed. This is followed by a description of the measures and the parking surveys, and also a discussion of the new situation. Finally, a number of conclusions are drawn.

PARKING SITUATION BEFORE INTRODUCTION OF MEASURES

Leeuwarden has about 85,000 inhabitants. The town center comprises the inner core, surrounded by canals and linked by eight bridges to the rest of the town and the station area. The center has about 2,800 inhabitants and a working population of more than 11,500 (1980 figures). It covers an area of about 900 x 1000 m. Figure 1 shows the exact boundaries.

The parking situation in the second half of the 1970s was considered unacceptable in several respects. In absolute terms, there was a shortage of parking space. Moreover, people working in the center were taking up parking areas intended for visitors and shoppers. The latter group tend to pay their calls in the second part of the morning, or in the afternoon, from 2:00 p.m. onwards. The working population, however, arrives earlier, both in the morning and in the afternoon, than the majority of visitors. The consequence is that visitors have to walk considerable distances or else park in places not intended for that purpose. The residents and tradespeople in the town center were also having problems. It was often difficult for them to find a parking place when they returned to their homes or business premises because any places reasonably close to their destination were taken by people working in or visiting the center.

PARKING MEASURES

The main objective of the parking plan (1) drawn up by the local authorities is to reallocate the number of parking places in a way that is attuned to the various categories, each with their own requirements

FIGURE 1 Leeuwarden town center.

as to the length of their stay. The reallocation is based on the following principles.

1. The residents of the town center and other interested parties should have parking facilities at their disposal close by.
2. Visitors to the town center should be able to park fairly close to their destination, especially if they only wish to park for a short time. The longer the parking lasts, the further the walking distance may be.
3. Long-term parking, particularly for those working in the center, should be provided on the periphery of the town center, in appropriate car parks.

The new system is intended to put a stop to the situation whereby the working population was able to take over the attractive free parking places at a short or medium distance from the shopping center. The old situation forced visitors to the town center to use the legal parking places at greater walking distance (just within the center or outside of it) or to park illegally. This in turn could have important consequences for visits to the town center, and consequently its long-term functioning.

The effect produced by the application of basic principles of the parking plan was to allow longer maximum parking time for parkers who have to walk the farthest to the core of the shopping center. Parking rates are also lower at parking places farther from the center. At the periphery of the center there are car parks where parking is allowed for a maximum of 1 day.

The series of parking measures introduced in Leeuwarden center in November 1979 include

1. An interested-parties arrangement;
2. A drastic increase in parking rates for places that were paid for in the old situation;
3. A drastic increase of paid parking; in the new situation, legal free parking in the center no longer exists;
4. Reduction of permitted parking time; and
5. A new underground car park with 625 places.

The interested-parties arrangement is an important element in the range of parking measures introduced. According to the by-law on parking and parking moneys, in which this arrangement is included, the burgomaster and alderman may issue a parking permit or season ticket to

1. The owner or licensee of a motor vehicle who lives in an area where interested-party parking places are located, and
2. The owner or licensee of a motor vehicle who has a business or exercises a profession in an area where interested-party parking places are located, and can prove that it is necessary, in the interests of this business or profession, to park the motor vehicle in such a parking place.

The permit or season ticket authorizes its holder to park in only one particular part of a street or one car park. Permits are only issued for streets reserved for residents and service traffic. In the past season tickets were supplied for car parks only. Since November 1980 a season ticket can also be obtained for a place at a parking meter. In 1980 the monthly rate for a parking permit or season ticket was Fl. 15.

Formerly, payment was required for about 1,200 of the 3,250 public parking places in the town center. Rates were doubled for 0.5- and 4-hr meters, and tripled for the 1-hr meters.

The data in Table 1 give the number of public parking places available in the old situation and now that the measures have been introduced. The number of parking places with payment increased from 1,200 to nearly 3,500. The 2,050 free places in the old situation have been converted into paid places.

TABLE 1 Public Parking Places in Town Center

Type of Parking Place	Hourly Rate (Fl.) 1978	Hourly Rate (Fl.) 1980	No. of Places[a] 1978	No. of Places[a] 1980
Limited parking				
0.5-hr meter	1	2	90	80
1 hr	0.50	1.50	150	310
0.5-1.5 hr (parking-disc zone)	-	-	330	-
2 hr	-	1	-	800
4 hr	0.50	1	970	290
Total			1,540	1,480
Public parking				
Ticket machine	-	1[b]	-	1,380
Underground car park	-	1.25	-	630
Free parking	-	-	1,720	-
Total			3,260	3,490

[a] Rounded figures.
[b] Per day.

The number of 4-hr meters has been cut considerably; some of the original places with unlimited parking have been changed to parking for a limited time. Parking in the underground car park, which is near the shopping center, is unlimited, although the car park is closed at night.

In addition to the measures regulating parking, a great many measures have also been taken relating to traffic circulation. One-way traffic has been introduced for most of the main streets in the center. Moreover, some streets are accessible to residents and service traffic only.

PARKING SURVEY

At the beginning of 1978 a plan was completed for Leeuwarden that contained measures regulating short-term parking (1). The compilers reached the conclusion that not enough was known about parking actions, in regard to the reason for and the duration of parking. Subsequently, the municipal authorities commissioned DHV Raadgevend Ingenieursbureau B.V. to carry out a parking study and shopper survey in the autumn of 1978 (2).

The new parking measures were introduced in November 1979. The desirability of examing their effects had already been noted. This became all the more pressing when the consequences of the measures became apparent: there was a substantial drop in parking in the center and the underground car park was almost empty. In 1980 the municipal authorities held a similar parking study of their own that, on the whole, ties up with the 1978 study (3).

In the autumn of 1981, a year after the second study, another limited follow-up study took place. At the request of the Project Bureau for Integrated Transport Studies, DHV analyzed the data obtained, adding to the information and analyzing it in more depth (4).

In June 1982 the evaluation of the parking measures was discussed in the appropriate committee of the town council. A few minor adjustments were decided on.

The studies at the end of 1978 and 1980 consisted of several parts. Parking occupancy was recorded, parking duration was observed, and a survey was held among people parking in the inner core and among people visiting the shopping centers on the outskirts of the town. A survey of pedestrians was also held at all points of entry into the center.

The results of an opinion poll by Lagendijk's research agency among the inhabitants of Leeuwarden were also used. This opinion poll included questions on traffic and parking in the town center. Van Heesewijk's Town and Country Planning Agency also collected data for a study of distribution and planning (5,6). Other data were also used, such as those relating to the development in the number of inhabitants and jobs, traffic censuses, and the costs and proceeds of parking.

POLICY QUESTIONS

The study carried out for the Project Bureau for Integrated Transport Studies, using all the available information, is intended to answer several questions relating to policy. These concern the effects of the parking measures, which can be subdivided into three groups:

1. Effects on parking behavior (primary effects),
2. Effects on mode of transport chosen (secondary effects), and
3. Effects on the functioning of the center's activities (tertiary effects).

The analysis relates almost exclusively to primary effects. There is insufficient information to permit conclusions on secondary and tertiary effects; moreover, the 10-month period between the initiation of the parking measures and the second study is too short to establish any tertiary effects. The policy questions are as follows:

1. Is there an improvement in ease of parking for residents and tradespeople?
2. Is there an improvement in ease of parking for shoppers?
3. Is there a change in the number of visits to the town center?
4. Is there a reduction in ease of parking for people working in the center?
5. Is there a change in the distribution pattern of parking congestion?
6. What function is the underground car park fulfilling with respect to parking in the center?
7. Has illegal parking increased?
8. Is there a change in the net proceeds of the parking system?

PARKING SITUATION AFTER INTRODUCTION OF NEW MEASURES

General

Parking density in the center has dropped: on Tuesday mornings by 35 percent, on Thursday evenings by 17 percent, and on Saturdays by 16 percent (see Table 2 and Figure 2). This drop is the result of fewer parking actions and shorter average parking times. On Tuesdays the number of parking actions dropped from around 9,500 to around 7,500; figures

TABLE 2 Parking Density

	Density
Tuesday afternoon, 12:30-5:30 p.m.	
1978[a]	3,550
1980[a]	2,300
Percent	-35
Thursday evening, 6:30-10:00 p.m.	
1978[a]	2,650
1980[a]	2,200
Percent	-17
Saturday, 10:30 a.m.-5:00 p.m.	
1978[a]	2,500
1980[a]	2,100
Percent	-16

Note: Density is the average numbers of parked cars, rounded off to the nearest 50.

[a]No. of parked cars.

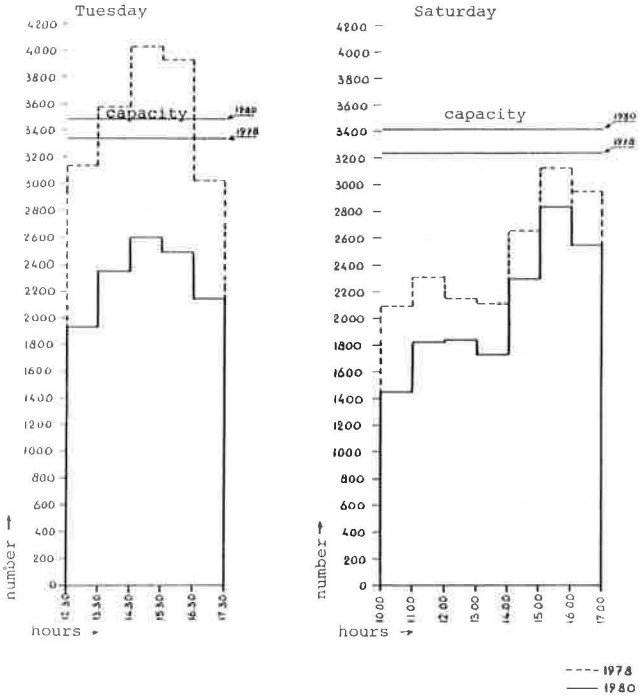

FIGURE 2 Parking density in Leeuwarden town center, 1978 and 1980.

for Saturdays are unchanged (around 10,000). Average parking time dropped on Tuesdays from 124 to 111 min, and on Saturdays from 88 to 79 min.

The data in Table 3 give parking density for different sections of the town center.

The data in Table 4 give the changes in the number of parking actions and the parking duration, according to the various parking motives. Apart from the motives of residence and business visit, parking has decreased substantially in the center for all motives. The greatest drop is among people working in the town center.

TABLE 3 Parking Density for Subareas (peak periods)

Area[a]	Tuesday, 2:30-4:30 p.m. 1978	1980
I	1.30	1.01
II	1.04	0.70
III	1.47	1.01
IV	1.39	0.95
V	1.22	0.43
VI	1.12	0.76
Entire center	1.22	0.73

Note: Density is the number of parked cars divided by the number of available public parking places.

[a]See Figure 1.

TABLE 4 Number of Parking Actions and Average Parking Time (Tuesday)

Motive	No. of Parking Actions[a] 1978	1980	Avg Parking Time (min) 1978	1980
Residence	1	1	179	113
Work	20	8	369	331
Shopping	47	36	85	56
Private visit	6	5	80	59
Business visit	10	11	116	68
Other	16	9	95	64
Total	100	70		

[a]Index 1978 = 100.

On Saturdays little has changed. The number of parking actions is almost the same, and there is only a slight shift in the share of the various motives in the total number of parking actions.

The data in Table 5 give an indication of the walking distances in the situation before and after the new policies were enacted. The average walking distance is considerably lower for all motives, except work. On Tuesdays this has even increased. The average walking distance, taking all the motives together, is 23 percent lower for Tuesdays and 9 percent lower for Saturdays.

In order to check how parking density had developed, another survey was held on a Tuesday afternoon in the autumn of 1981 (many shops are closed on Monday mornings or Wednesday afternoons). It proved to be 11 percent lower than at the end of 1980. This decrease is mainly due to a considerable reduction in illegal parking (about 90 percent). Occupancy of paid places was only down by 2 percent.

Residents and Tradespeople

Approximately 400 people make use of the interested-parties arrangement (29 percent residents, 37 percent resident tradespeople, and 34 percent tradespeople). Some 260 permits and more than 130 season tickets were involved. At the end of 1980 there were 519 interested-party places available. The local authorities only issue season tickets for car parks where occupancy does not exceed 0.85.

TABLE 5 Average Walking Distance (m) According to Motive

Motive	Tuesday 1978	1980	Saturday 1978	1980
Residence	–[a]	–[a]	–[a]	–[a]
Work (fixed address)	199	242	180	105
Shopping	279	214	273	245
Private visit	209	183	246	222
Business visit	235	154	232	68
Other	234	92	200	196
Avg	244	188	258	236

[a]Average walking distance is not known.

In view of the increased rates and restricted parking time for other parking places, interested parties are largely dependent on the interested-parties arrangement. This is the least-expensive alternative for people who wish to park near their home or business.

An opinion poll among the inhabitants of Leeuwarden by Lagendijk's agency in February 1981 (5) produced several interesting opinions on the measures regulating parking and traffic circulation that were introduced at the end of 1979. Altogether, 690 people were interviewed. Two-thirds of these people considered the drastic measures to be an improvement; there was little difference between car users and noncar users. Two-thirds also considered it right that parking in the center was no longer free. Sixteen percent of households with cars believed that the town center had become more accessible, as opposed to 36 percent who said it was less accessible. Thirty-four percent of the households that did not own a car (31 percent) found the center more accessible, whereas 14 percent found it less accessible. On the whole, parking rates were thought by car-owning households (53 percent) to be too high. About two-thirds of the respondents agreed that the new situation made the center more pleasant to walk in and to be in. They indicated that the use of cars for trips to the center had decreased.

It must be noted, with respect to these figures, that the opinion poll only involved inhabitants of Leeuwarden. The opinion of visitors to the town center from around Leeuwarden is not known. It is doubtful whether they share the inhabitants' opinions because they are far more dependent on transport by car.

Shoppers and Other Visitors

The number of shoppers parking in the center in the new situation has dropped 25 percent compared with the old situation. Use of parking places for visitors (to shops) has decreased substantially on Tuesdays, from 0.98 to 0.39. The walking distance for that group has also dropped considerably, from 279 to 214 m (on Tuesdays).

The available data do not indicate whether parking places are now being selected in the fringe around the center or whether visits (to shops) are being stopped, and to what extent the choice of mode of transport has changed. However, the Lagendijk opinion poll has revealed some shift in favor of cycling, but the extent is not known.

Number of Visits to Center

The research material does not provide a decisive answer to the question of whether there has been a change in the number of visits to Leeuwarden center. It is known that visitors to the shopping centers on the outskirts maintain that they visit the main shopping center less frequently (more frequently 3 percent, less frequently 24 percent).

People Working in the Center

In the old situation the motorists among the people working in the center, on arrival at work in the mornings, found enough parking space. In view of the limited duration of parking elsewhere, free places were preferred.

The parking situation in the town center has changed radically for this group with the advent of the new parking measures. The new situation requires payment for all parking places, and parking time is restricted as well.

The average walking distance for people working and parking their cars in the center has increased from 199 to 242 m (on Tuesdays). The number of people parking in the center, with the motive work, and their share in the use of places has dropped substantially. Their numbers have dropped 60 percent.

This decrease ties in with the objective of the measures. A quarter of this 60 percent (about 240) now park on the fringe of the center, although in the past they had parked in the town center. A quarter of the group now leave their cars on their own (business) parking lot in the inner city. It appears as though some people employed in the center have switched from private cars to public transport or bicycles. This cannot, however, be verified from the survey results.

Distribution Pattern of Parking Congestion

In 1978, in almost all parts of the center during the peak period, the degree of occupancy could be described as extremely high. In 1980 this no longer applied to about half of the inner core. In part of the town there is still overcrowding on Thursday evenings (late night shopping) and on Saturdays.

The total overspill to the fringe is not known. As was mentioned in the previous subsection, some of the people who work in the center no longer parked there after the measures were introduced, but instead left their cars on the periphery of the town center.

Illegal Parking

There are various forms of illegal parking:

1. Parking where there is a general prohibition (at bus stops, pedestrian crossings, entrances to premises, on street corners, and so forth),
2. Illegal use of interested-party places,
3. Insufficient or no payment, and
4. Exceeding the time limit.

The data in Table 6 give the parking volumes during peak periods in areas where a general prohibition applies. The drop in illegal parking, both in terms of absolute volume and as a percentage of total occupancy, is partly caused by increased checks. To ensure that the parking measures are properly observed, the number of parking wardens was gradually increased from 5 in October 1978 to 15 in September 1980. All wardens have the authority to fine parking offenders. Nevertheless, in 1980 illegal parking still comprised 20 percent of the total occupancy. Although a great many legal places are available, large numbers of cars are still parked illegally.

TABLE 6 Legal and Illegal Occupancy During Peak Periods

	Tuesday, 2:30-4:30 p.m.		Thursday, 7:00-9:00 p.m.		Saturday, 3:00-5:00 p.m.	
	1978	1980	1978	1980	1978	1980
Legal occupancy	2,850	1,900	2,200	2,000	2,300	2,200
Use of illegal places	1,150	600	800	450	700	350
Total	4,000	2,500	3,000	2,450	2,000	2,550

Note: Numbers are rounded off to the nearest 50.

Illegal use of places reserved for interested parties is as follows: more than 35 percent of the cars do not have an appropriate permit, and the total occupancy of those places is in fact only 60 percent; therefore, illegal parking there does not cause a shortage of space.

In 1980, 46 percent of the people using parking meters were in default of payment (Tuesday afternoons). The number of people exceeding the time limit has dropped a little.

Net Proceeds of Parking System

The annual proceeds from parking are 5 times higher since the new system has been operating (0.35 million guilders in 1979 and 1.8 million guilders in 1980). Costs are 5 times higher too (1.50 million guilders now as opposed to 0.3 million guilders previously). These sums do not include the proceeds and costs of the underground car park, nor have the costs of the land and of constructing the parking places been included.

In 1980 Fl. 75,000 of the proceeds were derived from charges to interested parties. The increase in costs is mainly generated by the extra parking wardens (5 in October 1978, 18 in January 1980, and 15 in September 1980).

Underground Car Park at Wilhelminaplein

When the 1978 study was being conducted, construction of the underground car park was already under way. Consequently, Wilhelminaplein was barred for parking at that time.

Before the car park was built the 400 parking places in the square were in great demand. Now it is no longer available for parking, but is reserved for activities like the weekly market, fairs, and so forth.

After the underground car park (close to the core of the shopping center) was opened in November 1979, it could be used free of charge for more than a month. During that period it was almost always fully occupied. Since parking charges have been introduced, at the rate of Fl. 1.25 an hour (up to a maximum of Fl. 10 a day), occupancy initially dropped considerably to 17 percent on Tuesdays, 40 percent on Thursday evenings, and 32 percent on Saturdays (1980 figures). The underground car park was mainly used for short- and medium-term parking. On Tuesday afternoons 70 percent of the people parked there for less than 2 hr, and on Saturdays 80 percent. On Thursday evenings, Fridays, and Saturdays 80 percent of the parkers give shopping as their motives for parking. On other days season ticket holders account for much of the occupancy (mostly people working in

the center). A monthly ticket costs Fl. 65. In 1980 about 60 of these tickets were issued; most were purchased by businesses for their employees.

In 1980 the neighboring 1- and 2-hr parking places were better occupied than the underground car park, although rates for the former are more expensive (about Fl. 1.75 an hour). Evidently people prefer to park on the streets or in parking lots to the underground car park.

The maximum rate was later reduced from Fl. 10 to Fl. 2.50 a day. This was done because visitors to the (shopping) center often do not park for more than 2 hr, and a low maximum rate might make the car park more attractive. This is to the advantage of people parking their cars for longer than 2 hr.

In 1981 occupancy on Tuesday afternoons was more than 30 percent; in 1981 on Saturdays and once or twice a week the underground park was completely full. This does confirm its importance for shoppers. In 1981, over the entire year, 30 percent more cars parked in the underground garage than in 1980. The proceeds from parking fees were practically 30 percent higher in 1981 than in 1980.

CONCLUSIONS

Since a series of parking measures were introduced, the number of cars parked in the town center has dropped substantially. This applies more on Tuesdays (the first full shopping day in the week), when people working in the center and those visiting the shops monopolize most parking spaces, than on Thursday evenings (late-night shopping) and Saturdays.

The average walking distance is considerably lower: on Tuesdays 188 m instead of 244 m and on Saturdays 236 m instead of 258 m. Average parking time has dropped too.

In Leeuwarden 260 permits and 132 season tickets have been issued under the interested-parties arrangement (1980 situation).

It would appear that it was easier for residents and tradespeople in the center to find a satisfactory parking place in 1980 than in 1978. If a motorist has a parking permit, competition with other groups of parkers (visitors, shoppers, and people employed in the center) for a space is a thing of the past.

Since the parking measures have been operating, visitors (shoppers) can park closer to their destination. Degree of occupancy of appropriate spaces for visitors has dropped considerably on Tuesdays and to a slightly lesser extent on Thursday evenings. However, in part of the center there is still high occupancy.

The number of shoppers parking in the center is down by 25 percent. Parking rates are considered in the main to be too high. It is not certain from the available data whether people have changed to parking on the fringe of the center or have changed their mode of transport or have stopped their visits (to shops).

Average walking distance for people working in the center has risen from 199 to 242 m (measured for people parking in the center). Some are now parking on the fringe. More people park in their own (business) parking lots. The research data suggest that some people who work in the center have switched from car travel to public transport or bicycles. However, there is no firm evidence for this.

The use of illegal places has almost halved but is still considerable, nonetheless.

Occupancy of the underground car park has increased by more than 30 percent in 1980 (on Tuesdays). On Saturdays and once or twice a week it is completely full (1981). On Thursday evenings, Fridays, and Saturdays 80 percent of the people who use the underground car park give shopping as their motive.

Parking proceeds have gone up from 0.35 million guilders in 1979 to 1.8 million guilders in 1980. Costs have risen from 0.3 to 1.5 million guilders.

The effects of the parking measures correspond largely with the municipal authorities' objectives in implementing this parking policy.

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Abridgment

Whither Parking in the City Center?

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ABSTRACT

The financing opportunities and options for providing new downtown parking are analyzed in the context of current fiscal realities. The present financial posture of typical municipal parking agencies is identified, the cost of providing new downtown facilities is analyzed, and means of obtaining needed revenues are suggested. Finally, complementary parking policies are suggested. Analysis of eight parking agencies in Middle Atlantic and New England states indicates average annual net incomes of about $125 per space. This compares with the $800 to $1,200 annual debt service outlay required for each new garage space. A cost-sharing concept is proposed based on the premise that parking produces benefits to many groups. Under this concept costs for new downtown garages would be shared among users, developers, the downtown community, and municipalities. Sound public finance principles that reflect a new fiscally accountable perspective are essential to parking in future city centers. This implies pooling of all parking-related revenues into a single fund, and increasing those revenues through rate adjustments, intensified enforcement, and better adjudication procedures. These funds would cover the costs of enforcement, operations, and, to the maximum extent possible, debt service. Corollary parking policy guidelines call for following rather than anticipating development; underbuilding rather than overbuilding; constructing smaller, simpler garages rather than megastructures; and reorienting downtown zoning requirements to actual needs.

Conveniently located, attractively designed, and reasonably priced off-street parking is vital to the downtown economy. Yet in most cities it has become clear that downtown parking can no longer pay its own way through parking revenues. The financing opportunities and options for providing automobile parking are analyzed in this paper. The current financial posture of typical municipal parking agencies is identified, the costs of providing new facilities are analyzed, and means of obtaining needed revenues and meeting expenses are suggested. Parking development and management policies, which are keyed to the fiscal realities of the 1980s, are also suggested.

CONTEXT

The high costs of developing, maintaining, and operating parking space make it increasingly more difficult to provide financially self-supporting parking facilities. Parking fees are often insufficient to cover the debt service; frequently they are little more than what is required to meet day-to-day operating costs. Few parking operations are financially able to establish enough capital reserve to cover expansion or major repairs. At the same time, most cities are caught in a cost/revenue squeeze, relative to the services they must provide and the resources that are available. Thus subsidizing the development and operation of parking is rapidly becoming a less-affordable proposition for these communities.

The magnitude of the problem of affording new parking development varies, depending on city or local jurisdiction. Larger cities with rail transit are placing limits on the amount of central business district (CBD) parking that can be provided. High demand for the limited supply of parking space in these cities means that relatively high parking fees can be charged and, paradoxically, new parking facilities can be financially attractive, if permitted. Some cities require developers to share in the cost of parking, but many more cities are reluctant to follow this policy in the CBD for fear of discouraging development. For nearly all cities, the challenge is how best to attract new development while minimizing the parking cost to the city.

FINANCING NEEDS AND RESPONSIBILITIES

Typically, parking revenues barely cover operating costs, leaving relatively little reserve for either major maintenance or parking system expansion, especially after debt service obligations are considered. The comparative parking capacities, revenues, operating costs, and net revenues for eight medium-sized New England and Middle Atlantic cities, summarized in Table 1, illustrate the problem. Annual revenue per space ranges from $214 to $580, and averages $373. Annual net operating income before debt service ranges from $3 to $287 per space, and averages $127.

The last figure is especially significant because it indicates the limited reserve available for system expansion. It also indicates the likely finan-

TABLE 1 Comparative Financial Performance of Parking Agencies in New England and Middle Atlantic Cities

City	Spaces in System[a]	Annual Revenue per Space ($)	Annual Maintenance and Operating Cost per Space ($)	Annual Net Income per Space ($)
1	11,300	361	228[c]	133
2[b]	4,100	446	443	3
3	3,600	349	205	144
4	2,300	580	293	287
5	1,900	214	160	54
6	1,300	323	180	143
7	1,200	474	271	203
8	1,200	240	190	50
Mean	3,360	373	246	127
Standard deviation	3,390	122	91	91

Note: Data are from parking agency annual reports from New Haven, Stamford, and Waterbury, Connecticut; Wilmington, Delaware; Worcester, Massachusetts; Paterson and Trenton, New Jersey; and White Plains, New York.

[a] Numbers are rounded.
[b] Data for CBD spaces only.
[c] Metered operation.

cial performance of new parking space. Net operating income can be compared with the annual debt service cost of $800 to $1,200 per space, from which the annual subsidy per space can be estimated.

The actual amount of subsidy per parking space depends on the development costs and interest rate, as balanced by income. Figure 1 shows how the annual subsidy needed to break-even rises as parking development costs rise and net operating income declines. It is based on a 10 percent interest rate and 30-year debt service.

At 1983 cost levels, garage development expenses, including land, construction, engineering, legal, and contingency costs, approximate $10,000 per space. Net operating incomes range from $50 to $300 per space (Table 1). This results in annual subsidies (deficits) of $760 to $1,060 per space. Even with increased rates and greater operating efficiency, user charges will not cover debt service costs under these circumstances. Revenue bond financing may not be possible with debt service coverage ratios as high as 1.2 or 1.3. Financing for new parking usually requires broader financial backing than that obtainable from parking revenues alone.

Parking development costs should be shared among users, the municipality, and specific beneficiaries according to some predetermined basis. Users, for example, should cover operating costs plus a specified minimum portion of development costs. The remaining capital costs would be distributed among the city, downtown benefactors, and designated developers. The use of tax increment financing to pay debt service is another possibility. The cost-sharing concept recognizes that parking is a public service. Therefore, it is reasonable to expect each group that benefits to share the costs of new parking development. It is an application of the well-established public finance principle of cost recovery.

Although cost sharing is correct in principle, it may prove difficult in practice. Many parking authorities do not have the financial reserve to cover outstanding portions of their debt service. Local government limitations on debt service obligations may prevent contributions from the general fund. Moreover, some cities already have assumed as much debt service obligation as they can afford to bear. For these cities other complementary approaches to parking facility finance are needed.

STRATEGY FOR SYSTEM FINANCE

A systems approach should be adopted for all aspects of parking facility development, operation, and finance. This approach should reflect sound business and public financing principles, should enhance the financial integrity of public parking agencies, and should reduce the reliance on general municipal financing.

All on-street and off-street parking-related revenues should be pooled into a single parking fund. The revenue sources normally include municipal lot and garage revenues, curb parking meter revenues, license fees paid by private operators, and parking ticket fines. The fund should include any additional revenues that result from rate increases, more intensified enforcement, and improved ticket adjudication procedures. The parking fund should be used to cover the costs of enforcement, operation, and, to the extent possible, debt service. The parking fund should not be used to subsidize other public needs.

Parking agencies must be fiscally accountable for their actions. They should adjust parking fees to what the traffic will bear. At a minimum, rates should keep pace with inflation. They should search for ways to maximize income through the use of empty space during off-peak times.

Based on 1983 dollars, operating expenses should not exceed $350 to $375 per space. Where unit costs exceed these values, efforts should be made to cut costs. Typical areas for cost reduction include (a) reducing hours of operation, (b) trimming operating personnel and administrative staff, (c) insuring on the outstanding debt rather than on full replacement value, and (d) consolidating or relinquishing small facilities. In some cases private operation by means of management contracts secured through competitive bidding can reduce costs.

PLANNING AND POLICY GUIDELINES

Downtown parking programs should recognize parking as an important public service that benefits users, businesses, and the general community. The pattern, placement, and size of new parking facilities should be designed to reinforce commercial activity and downtown development projects, and should reflect the economic realities of the community. Parking

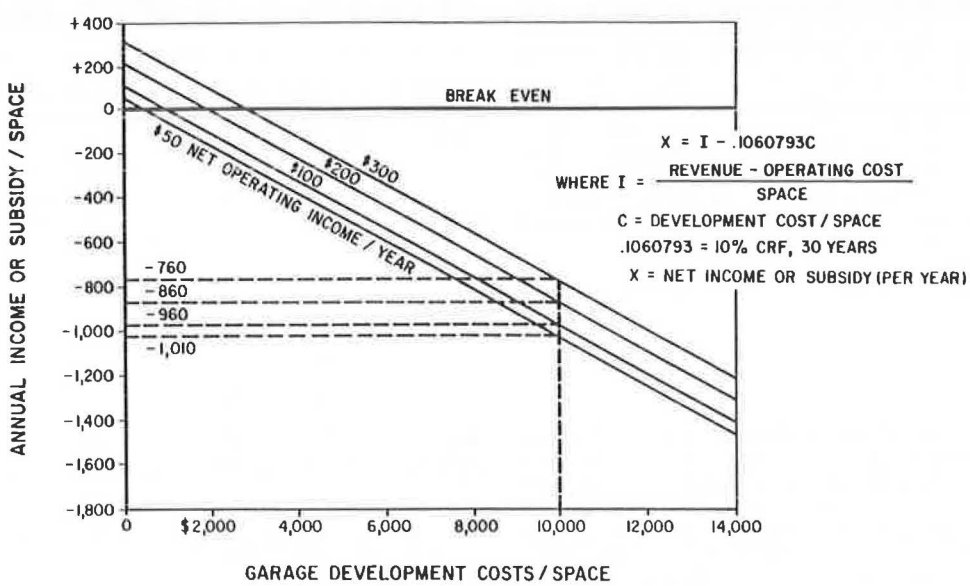

FIGURE 1 Parking subsidy per space.

policies should consider the unique needs and resources of the community and should encourage private-sector responsibility. Within this context, the following guidelines complement the system finance proposals.

Underbuild Rather than Overbuild

New downtown parking facilities should be coordinated with developments that are in progress or in advanced planning. They should not lead development, as was often the case in the past. Most cities do not have the resources for anticipatory or speculative parking developments. The goal should be to provide as little new parking as needed to attract new investment. Cities should underprovide parking relative to future demands. A philosophy of selective underproviding has two benefits: it helps to maximize use of existing parking and it reduces parking development costs.

Give Priority to Short-Term Parking

Parking problems should be met in order of importance. Normally this means providing for business and shopping trips first, before work trip commuters. All short-term parking demands should be provided for, plus a specified portion of work trips. In smaller cities, such as New Haven and Providence, it may be appropriate to provide for up to 75 percent of the long-term work trip parking demand. The proportion should be reduced as downtown size increases, so that in large cities only short-term parking demands are met. (In large cities that have extensive rail transit it may be desirable to continue to limit future parking supply.)

Locate and Design for Maximum Value

New parking should be located within convenient walking distance of the activities they serve. Walking distance generally should not exceed 500 ft for short-term parking and 1,000 ft for long-term parking. Parking facilities should not be located so as to compete with existing facilities. Parking design should not inhibit or discourage users, and because parking structures have a long service life, they should be built to design standards that will remain adequate over the projected life of the facility.

Size New Parking for Economy of Operation

Priority should be given to the staged development of 500- to 750-space project-related garages. Experience suggests that this size range of garage is the most economical to operate. Smaller facilities should be relinquished or consolidated where possible.

Consider Ground Floor Commercial Space

Joint development of ground floor commercial space should be encouraged. The exceptions are where it precludes efficient garage design, substantially increases construction costs, or results in unproductive or inefficient retail or commercial space.

Minimize Parking Development Costs

Minimizing costs calls for avoiding complex design, difficult sites, and elaborate architectural treatments. Existing architectural controls should be reassessed with a view toward reducing costs.

Control Commercial Parking Operations

Municipalities should exercise control over commercially operated lots and garages with respect to location, design standards, and operating procedures. Ideally, rate ranges should be set for downtown rate zones based on proximity and land use.

Avoid Excessive Parking Requirements

Parking space requirements for downtown land use should reflect actual needs. They should consider factors such as floor space to employee ratios, car occupancies, transit service availability, and the interactions among downtown land uses. In cities with rail transit systems, zoning standards should specify both minimum and maximum requirements for each type of activity based on proximity to transit stations.

PROSPECT

Whither parking in tomorrow's city center? How can it keep pace with downtown change? What directions should its planning, management, and financing take? These questions are among the issues addressed in this paper.

Downtown parking is an important urban land use and a vital public service. Parking policy should be attuned to each city's development prospects and financial realities. The key guidelines are to (a) follow rather than anticipate new development; (b) underbuild rather than overbuild; (c) construct smaller, simpler, less costly facilities rather than megastructures; and (d) reorient, and possibly reduce, downtown zoning requirements.

Cities have a major investment in downtown parking. Where land uses change (i.e., retail decline), cities should capitalize on underused parking that is already available in planning new development.

Sound public finance principles that hold parking operations fiscally accountable are essential. Greater attention must be given to attracting and effectively working with the private sector in parking development and financing. Such arrangements must provide opportunities for both the public and private interests to share in the rewards, as well as the risks. Cities should pool all parking-related revenues into a single dedicated parking fund, and increase these revenues through rate adjustments, intensified enforcement, and better ticket adjudication procedures. The fund should cover the costs of enforcement, operation, and, to the maximum extent possible, debt service.

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Contraflow Bus Lanes in Chicago: Safety and Traffic Impacts

JOHN LaPLANTE and TIM HARRINGTON

ABSTRACT

Contraflow bus lanes were installed on the downtown portions of four Chicago streets in 1980 and 1981. They were installed as a part of a federally mandated air quality improvement program and also to increase bus patronage by improving east-west bus service reliability across the central business district. Since their installation there has been public concern about the pedestrian safety aspects of the bus lane operation as well as increased traffic congestion on the remaining lane street space. Regarding vehicular congestion, it was found that nonbus traffic did travel somewhat more slowly with the installation of the lanes. However, in terms of bus operations, there was a significant improvement both in average travel speeds (22 percent increase) and in service reliability, which resulted in annual cost savings to the Chicago Transit Authority of about $400,000. Patronage studies indicate that during peak periods the contraflow lanes move more people in one lane per street than are moved on the remaining through traffic lanes, and in only one-tenth of the number of vehicles. Concerning the safety of the bus lane operation, the actual accident data indicate that although there was an initial jump in bus-pedestrian accidents, these accidents have now returned to less than one additional accident per month on each street, and that there has been a 19 percent overall decrease in all pedestrian accidents and a 52 percent decrease in all bus accidents. The primary conclusion is that the bus lanes are operating in a safe and effective manner, and it is recommended that they be retained.

The City of Chicago installed contraflow bus lanes on the downtown portions of Adams Street and Jackson Boulevard on August 31, 1980. A second bus lane pair was installed on Washington and Madison streets on September 13, 1981. The contraflow bus lanes on Adams, Jackson, and Washington extend from Michigan Avenue west to Jefferson Street, whereas the Madison bus lane extends west to Desplaines Street (see Figure 1).

Bus lane implementation resulted, in part, from the 1973 U.S. Environmental Protection Agency (EPA) regulations promulgated to reduce air pollution in the central business district (CBD) of Chicago. This federally mandated air quality improvement program included both implementation of CBD parking restrictions and the contraflow bus lanes. For the parking restrictions, 10 one-way downtown streets had parking completely prohibited along one side. Tow-away zones were instituted on these 10 streets between

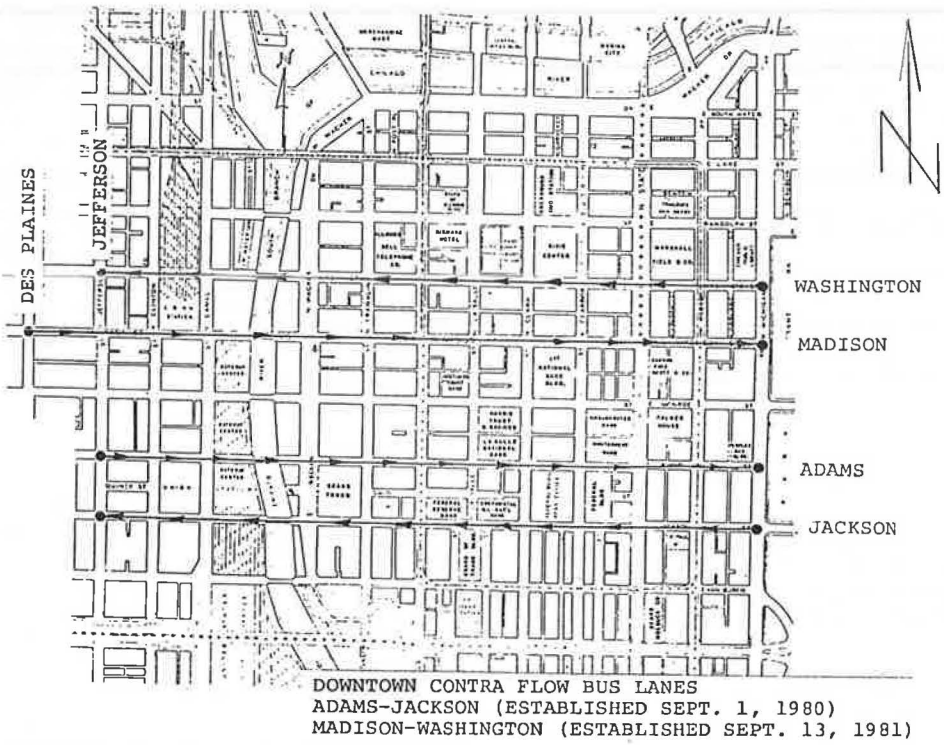

FIGURE 1 Map of contraflow bus lanes.

November 1974 and July 1975. The exclusive bus lanes, the second part of this program, were authorized by the Chicago City Council through an ordinance passed on July 7, 1977.

Response to the federal mandate was neither the sole reason nor the most important reason for implementation of the contraflow bus lanes. Historically, bus service crossing downtown had been unreliable, including that between the commuter rail stations west of the Chicago River (Union Station and North Western Station) and sections of the CBD to the east. Buses were subject to delays encountered in CBD traffic, which at times were substantial, and reliable schedules often could not be maintained.

Before the installation of the contraflow bus lanes buses operated on all east-west downtown streets with the flow of traffic. The majority of buses loaded and unloaded passengers on the near side of each intersection at the right-hand curb. These buses then moved into the second lane to pass parked vehicles in proceeding to the next stop. Conflicts between buses and right-turning vehicles occurred at virtually every stop because of pedestrians blocking turning vehicles or buses engaging in passenger interchange. In addition, the Chicago Police Department found it difficult to strictly enforce the No Parking Tow-away Zone indicated on one side of these one-way streets.

In the late 1950s Chicago experimented with a same-direction bus lane in the middle of Washington Street in the CBD, but this operation was discontinued in the 1970s because of difficulties in keeping the lane free of nontransit traffic, severe congestion on the remaining travel lanes because of the elimination of the center lane of travel, and safety hazards with the midstreet passenger loading islands.

In an effort to reduce or eliminate these problems, contraflow bus lanes were established. These bus lanes were implemented in two pairs, each pair providing east-west bus service across the Loop and serving the commuter rail stations on the west end. The south bus lane pair, Adams and Jackson, consolidated nine bus routes onto two streets; the north bus lane pair, Madison and Washington, consolidated seven routes onto these two streets. This consolidation has removed nearly all buses from other east-west Loop streets.

PROBLEMS

Each of the four bus lane streets is an east-west one-way street, with transit vehicles operating in the opposite direction. One curb lane on each bus lane street is exclusively used for contraflow transit vehicles. These reserved bus lanes and the adjacent painted median that separates the bus lanes from regular traffic are up to 4 ft wider than the parking lanes they replaced.

The operation of the contraflow bus lanes gave rise to two public concerns: the safety of bus lane operation, and the traffic congestion on bus lane streets. The public concern about bus lane safety relates to perceived pedestrian hazards at the Loop. The lack of awareness of the bus lane's location (immediately adjacent to the sidewalk) and the direction of bus traffic opposite to the direction of other vehicular traffic could cause possible safety problems to downtown pedestrians. The public's concern about traffic congestion on bus lane streets stemmed from initial slower traffic speeds and goods delivery problems on bus lane streets, when viewed next to the often empty bus lanes.

BACKGROUND OF EVALUATION STUDY

In response to these perceived problems, two Chicago aldermen submitted a resolution to the Chicago City Council on June 13, 1982, "to cease the use of reverse-flow bus lanes and reinstitute the previous system of center lane bus lanes." This ordinance was referred to the Chicago City Council Committee on Traffic Control and Safety, which directed the Mayor's Traffic Management Task Force to undertake a comprehensive evaluation of the safety and traffic impacts of the contraflow bus lanes.

A Traffic Management Task Force Contraflow Bus Lane Subcommittee was formed that included representatives of the city departments of Police, Public Works, and Streets and Sanitation; the Chicago Transit Authority (CTA); the Chicago Area Transportation Study; the Chicago Association of Commerce and Industry; and the Chicago Central Area Committee.

TRAFFIC OPERATIONS ANALYSIS

Vehicular Congestion

The primary objectives of the contraflow bus lanes were to decrease bus travel times and to improve bus schedule regularity. The bus lanes were implemented, noting that a negative effect on nonbus traffic speeds could result, and nonbus traffic during just the peak periods has slowed somewhat since the installation of the lanes. This reduction in travel speeds was particularly severe immediately after the lanes were installed, but recent efforts by the Mayor's Traffic Management Task Force have significantly lessened these adverse traffic impacts.

Specifically, when the lanes were first installed, traffic speeds during all downtown traffic peak periods (morning, noon, and evening) dropped precipitously. Comparison of data gathered in 1975 and 1981 (see Table 1) indicate that average traffic speeds on the bus lane streets fell from 8.41 to 4.34 mph, a 48 percent drop. Another way of viewing these statistics is to translate these average speeds into average travel times for the trip across the CBD between Michigan Avenue and the Chicago River. In 1975 this trip on these four streets took an average of 5.0 min, whereas in 1981 the average time for the same trip on the same streets was 9.7 min, a 94 percent increase. (There was little added congestion during nonpeak periods.)

A number of factors created this reduction in travel speed during peak travel times. First, the

TABLE 1 Contraflow Bus Lane Streets: Average Automobile Speeds

Street	Avg Automobile Speed (mph)					Change from 1981 (%)
	1974	1975	1980	1981	1982	
Adams						
Morning	9.61	8.09	6.75	5.22	9.66	+85
Noon	8.75	7.91	5.71	3.76	7.07	+88
Evening	9.40	8.58	6.48	2.71	7.25	+167
Daily avg	9.25	8.19	6.31	3.90	7.99	+104
Jackson						
Morning	8.30	7.05	6.38	3.49	6.78	+94
Noon	8.51	5.80	5.12	3.17	6.58	+108
Evening	6.28	5.68	4.10	3.81	6.62	+74
Daily avg	7.70	6.18	5.20	3.49	6.66	+91
Washington						
Morning	9.20	7.60	-	4.15	5.87	+41
Noon	5.99	6.08	-	4.93	9.13	+85
Evening	6.38	6.28	-	5.83	7.84	+34
Daily avg	7.19	6.65		4.97	7.61	+53
Madison						
Morning	10.03	14.57	-	7.01	11.61	+66
Noon	7.16	11.50	-	4.76	6.21	+30
Evening	9.86	11.83	-	3.21	8.71	+170
Daily avg	9.02	12.63	-	4.99	8.84	+77
Total avg	8.29	8.41	5.76	4.34	7.78	+79

impact of delays made by turning vehicles was exacerbated by the bus lane occupying one traffic lane for exclusive bus use, thereby leaving only two lanes available for nonbus traffic. Heavy CBD pedestrian traffic delays a turning vehicle and consequently blocks through traffic in that lane. Where a bus lane street crossed a two-way north-south street (Wabash Avenue, LaSalle Street, Wells Street, Franklin Street, or Wacker Drive), both of the through traffic lanes could be delayed by pedestrian traffic blocking turning vehicles in each of these lanes, thus consequently blocking all nonbus traffic movement on the bus lane street. (Before the installation of the lanes there were often three lanes available at these intersections, which left at least one lane for through traffic.)

These blockages have been all but eliminated in the last year through actions taken by the Traffic Management Task Force. One-way traffic flows have been instituted on Franklin Street, Wells Street, and Wabash Avenue, and peak-hour turning restrictions have been added as needed on LaSalle Street and Wacker Drive. As a result of these actions at least one lane at every intersection on bus lane streets in the CBD remains open for through traffic movement.

A second cause of these lower traffic speeds in a few locations was CTA bus travel with the prevailing flow in the regular traffic lane. To accommodate the bus lanes, most nonbus lane widths were reduced to 9 ft. The presence of even a few CTA buses in narrow 9-ft lanes inhibits the flow of traffic on those and adjacent lanes. The Task Force has worked closely with the CTA, a member agency, to eliminate as many of these with-the-flow bus movements as possible.

A third reason for traffic delays was the occasional double parking by delivery vehicles. Formerly, if a loading zone or metered parking space was not available, a delivery vehicle would temporarily use the tow-away zone while making a delivery, thus blocking only one of the three available through traffic lanes. However, after the installation of the bus lanes in this former tow-away zone, a double-parked vehicle blocked one of only two available traffic lanes. The recent work of the Traffic Management Task Force in ensuring better on-street parking enforcement, as well as cooperative measures with building owners and managers to develop more efficient use of the existing off-street loading areas, has reduced but not eliminated this problem.

The final, and perhaps most important, factor influencing traffic speeds was the occasional roadway narrowing that occurred at building construction sites and utility repair locations. Construction activity in both 1980 and 1981 appears to have been a critical factor in explaining much of the decrease in travel speeds on bus lane streets. Because the congestion caused by these activities was one of the prime motivating factors in the creation of the Traffic Management Task Force, the coordination of these activities was a priority item when the Task Force was formed in June 1982.

The results of these coordination efforts (and the other traffic management efforts mentioned previously) have had a beneficial impact on traffic speeds throughout the CBD, and particularly on the four bus lane streets. Speed-and-delay runs taken in 1982 indicated an average increase in travel speeds of 79 percent on Jackson, Adams, Madison, and Washington when compared to 1981 (see Table 1). In terms of travel time, it currently takes an average of 5.4 min to cross the CBD in a regular traffic lane, which is only 0.5 min more than the 1975 travel time.

The data in Table 1 give average travel speeds in October 1974 (before the installation of the tow-away zones and parking meters), in November 1975 (after these measures), in October 1980 (after installation of the Jackson-Adams bus lanes), in October 1981 (after the installation of the Madison-Washington bus lanes), and in October 1982 (after 6 months of operation of the Mayor's Traffic Management Task Force).

During this entire before-and-after period of bus lane implementation, there were no significant traffic signal timing changes in the CBD. This is because the City of Chicago currently has no real control of the coordination of traffic signals within the CBD. In 1976 a computer-controlled CBD signal system was installed, using an experimental signal system program and newly developed equipment. This program did not work as promised, and the equipment manufacturer ceased manufacturing replacement parts. Thus the computer control has proved to be completely useless, and equipment malfunctions have prevented communication with and coordination of the various signalized intersections. The city is now in the midst of a feasibility study to determine what would be the best and most efficient signal system to use to replace the present system. However, this means that during the entire time period of this particular study, all of the downtown traffic signals operated independently of one another, and no signal timing changes other than directional split were studied or even contemplated with respect to the contraflow bus lanes.

Bus Operations

Implementation of the contraflow bus lanes has had a positive impact on bus operations across the CBD. The nine bus routes that use the Adams-Jackson bus lane pair and the seven routes that use the Washington-Madison pair have benefited from the exclusive transit street space. The average time a bus takes to cross the CBD (between Michigan Avenue and the Chicago River) during the evening rush period has decreased from 10.25 to 8.00 min, a 22 percent improvement, since the lanes were installed.

In addition, because the contraflow bus lanes provide a virtually congestion-free route across the Loop, the reliability of bus service has also improved. People waiting for bus service are now more confident that buses will be evenly spaced at their pickup point and that long waits for buses will not occur. Also, the consolidation of virtually all east-west bus routes onto the two pairs of contraflow lanes assures frequent bus service across the Loop. Consequently, passengers rarely have to wait for more than a couple of minutes to board a bus for a cross-Loop trip. Once on the bus, the 55,000 passengers who travel these bus lanes daily can expect the trip to their downtown destination to be made in consistent times from one day to the next.

This improvement in service reliability, however, also benefits all passengers on these routes, whether or not they ride in the Loop area. A far greater proportion of buses arrive at their downtown destinations on time or within the additional period of time provided for schedule recovery. Consequently, most buses are able to begin their outbound trip from downtown on schedule and, particularly in light of a congestion-free return trip across the Loop, are able to avoid the bunching of buses that is characteristic of routes that are subject to heavy street congestion. An orderly, evenly spaced flow of buses leaving downtown translates directly into better service in outlying areas and benefits each of the more than 200,000 daily passengers that ride routes that use the downtown contraflow bus lanes.

The combination of decreased downtown travel time and improved reliability results in reduced round-trip travel time, and allows CTA to provide the same level of service with fewer total buses and operators. A five-bus reduction in vehicle requirements has been realized, which yields an annual operation cost savings of about $400,000.

Patronage

The contraflow bus lanes were designed to provide fast and reliable east-west bus service in the Loop. The reservation of one full lane on each bus lane street, however, decreased street space for remaining automobile and truck traffic. Such reservation of public space for exclusive use raises questions of both equity and efficiency; namely, are enough people served by transit on the bus lanes to justify removal of automobile and truck traffic?

On an average weekday 123,000 persons are transported in cars, taxis, trucks, and buses over the four contraflow bus lane streets. Of these 123,000 persons, 55,000 (45 percent) are served by buses on the contraflow lanes (see Table 2). Use of the contraflow lanes does have peaking characteristics, however, that make the lanes more efficient during periods of heaviest traffic. Patronage figures were estimated from October 1982 traffic counts and CTA ridership data that give the number of people served at critical periods of the day.

TABLE 2 Persons Transported on Bus Lane Streets

	Traffic Lanes		Bus Lanes		Total, All Lanes
	No.	Percent	No.	Percent	
Daily (24-hr)	68,000	55	55,000	45	123,000
Morning peak period (6:00-9:00 a.m.)	10,950	41	15,500	59	26,450
Evening peak period (3:00-6:00 p.m.)	14,150	46	16,750	54	30,900
Morning peak hour	4,700	38	7,600	62	12,300
Evening peak hour	5,000	39	7,950	61	12,950

During the morning and evening peak periods, when the loss of one lane to automobile and truck traffic has its most significant effect on automobile and truck traffic flow, the contraflow lanes move more people in one lane per street than are moved on the remaining two or three through traffic lanes, and in only one-tenth of the number of vehicles.

Contraflow Bus Lane Survey

As part of the contraflow bus lane evaluation, surveys were conducted to determine the attitudes of bus lane user groups (CTA drivers and passengers), nonbus lane user groups (taxi, truck, and automobile drivers), police officers assigned to Loop traffic duty, and pedestrians. The attitudes of persons who use or who are affected by the bus lane streets are important for two reasons: (a) these attitudes can influence travel behavior, and (b) they are one measure of the success or failure of the bus lanes. Survey questions sought information on people's attitudes toward bus lane effects on both traffic efficiency and safety. (Survey data on the safety impacts will be presented in the next section of this paper.)

A summary of survey responses on traffic effects for each of five groups--police, CTA personnel, bus passengers, drivers, and pedestrians--is given in Table 3. The driver group is subdivided into automobile, taxi, and truck drivers. [A more complete report on these surveys can be found elsewhere (1).]

Those who benefit directly from bus lane implementation (bus drivers and instructors and bus passengers) have an overwhelmingly positive attitude about the bus lanes. Not surprisingly, those toward whom the bus lanes and their benefits were not directly targeted (automobile, taxi, and truck drivers) found the lanes to be inconvenient, but they were not as strong in their expression of disfavor as the bus-related group were in their positive expression. Both police officers responsible for directing traffic in the Loop and pedestrians who use these streets were positive about the lanes. Discussion of each group's responses follows.

Chicago Police Department

Sixty-three Chicago Police Department officers completed the bus lane questionnaire. Each of these officers is either currently responsible for on-street traffic direction in the Loop or supervises such activity. Police Department personnel perceived the bus lanes to be an advantage by almost a 5-to-2 margin. The greatest concern expressed about traffic issues was related to enforcement, with 21 percent of officers commenting on illegal use of the lanes, automobile and truck parking in the bus lane, jaywalking, delays caused by turning buses, or the need for tow zones on bus lane streets. Forty-six percent of those officers surveyed believed that the lanes saved time for buses, whereas 17 percent believed that the lanes saved time for automobiles and trucks. Only 24 percent believed that the lanes lost time for automobiles and trucks.

Chicago Transit Authority

CTA drivers now working the contraflow lanes and CTA instructors were extremely supportive of the bus lanes. Positive responses, 89 percent of the total

TABLE 3 Contraflow Survey Responses—Traffic Effects

	Police		CTA Drivers and Instructors		Bus Passengers		Drivers: Automobile		Drivers: Taxi		Drivers: Truck		Pedestrians	
	No.	Percent	No.	Percent	No.	Percent	No.	Percent	No.	Percent	No.	Percent	No.	Percent
Total respondents	63		636		334		222		50		50		617	
General perception of bus lanes														
Good	41	65	568	89	257	77	89	40	11	11	18	36	315	51
Bad	17	27	22	3	57	17	104	47	33	66	27	54	197	32
Time savings														
Bus	29	46	474	75	210	63	-		-		-		-	
Automobile	11	17	105	17	-		67	30	6	12	6	12	-	

636 operating and supervisory personnel surveyed, included drivers who perceived a time savings or who specifically indicated the lanes were good or should be retained. Only 3 percent believed that the lanes were bad or should be removed. Limiting responses to drivers who specifically called for the retention or abandonment of the lanes, 82 percent called for retaining the contraflow lanes. Nearly one-quarter (23 percent) of the respondents believed that automobiles saved time or benefited from the bus lanes. Ninety-five (15 percent) of the CTA personnel called for greater traffic control at construction sites and intersections, whereas 29 (5 percent) called for the installation of bus lanes on other central area streets.

Bus Passengers

Sixty-nine percent of the 334 bus passengers interviewed use a bus lane street every day. Slightly more than three-fourths (77 percent) were on business trips, and 67 percent had traveled on the street before. An overwhelming majority (77 percent) perceived the bus lanes as beneficial, whereas only 16 percent felt they were bad, and 7 percent had no opinion. A time savings was indicated by 63 percent of the persons interviewed.

Automobile Drivers

Sixty-four percent of the 222 motorists interviewed use a bus lane street daily. Eighty-eight percent of the motorists had used the street before, and 83 percent were on business trips. Only 40 percent of all motorists surveyed perceived the bus lanes as good, 47 percent indicated that they were bad, and 13 percent had no opinion. Even though 47 percent believed that the bus lanes were bad, 30 percent indicated that they saved time. Seventeen percent of the automobile drivers believed that the lanes caused delay or should be removed. Nine percent specifically commented that implementation of the lanes improved traffic flow.

Truck Drivers

Fifty truck drivers were interviewed at both on-street and off-street locations (52 percent off and 48 percent on). Seventy percent indicated that they made 26 or more deliveries per week. Ninety-six percent had used the streets before the implementation of the bus lanes, and 58 percent believed that they affected deliveries. Although 36 percent of the drivers had a favorable impression, only 12 percent indicated any time savings. A majority of the drivers (54 percent) indicated that they were delayed, and they believed that the bus lanes were bad.

Taxi Drivers

The 59 taxi drivers interviewed indicated that 88 percent of them worked 5 or more days per week, and the same percentage indicated that they were driving in the Loop before the implementation of the bus lanes. Most of the drivers (66 percent) generally did not like the bus lanes; however, 22 percent had a good impression and 12 percent had no opinion. Twelve percent of the drivers saved time, 76 percent were delayed, and again 12 percent had no opinion.

Pedestrians

Seventy-six percent of the 617 pedestrians interviewed used a bus lane street every day. Almost three-fourths (72 percent) of the pedestrians were on business trips, and 88 percent had used the street before the implementation of the bus lanes. A majority of all pedestrians surveyed (51 percent) perceived the bus lanes as good, 32 percent believed that they were bad, and 17 percent had no opinion.

General Observations

The combined survey activities of the Chicago Police Department, the CTA, and the Chicago Area Transportation Study resulted in 1,972 interviews. The majority (66 percent) had a good opinion of the bus lanes, whereas 23 percent believed the lanes were bad. The part of the survey that questioned the general public resulted in 1,273 completed interviews, with good opinions on the contraflow lanes outnumbering bad by a 54 to 33 percent margin.

Although the majority of respondents believed that the lanes were good, the attitude split along user group lines is important to note. All bus lane user groups (bus passengers and CTA personnel) believed the bus lanes were good, whereas all driver groups that use nonbus lane street space had a generally negative opinion. Both groups that did not specifically use the lanes for transportation (police officers and pedestrians) had a majority good opinion of the lanes. The perceived time savings attitudes aligned in the same way, with the majority of bus users and a minority of drivers noting time savings for the trip across the CBD.

TRAFFIC AND PEDESTRIAN SAFETY ANALYSIS

Contraflow Bus Lane Survey

As noted previously, the contraflow bus lanes have been perceived by some as being a pedestrian safety problem. One of the goals of the various bus lane surveys was to try to measure the extent of the perception that, as one respondent stated, "These lanes are great for buses, but are really dangerous for pedestrians."

However, when all the comments of pedestrians and other users of bus lane streets are tabulated, this perception is clearly in the minority. For example, of the 617 pedestrians surveyed, only 19 percent perceived the bus lanes as dangerous, and another 3 percent termed them confusing. Similarly, only 14 percent of the bus passengers perceived the lanes as dangerous or bad for pedestrians. Surprisingly, only 5 percent of the motorists, taxi drivers, and truck operators, who tended more toward opposition to the lanes, thought the bus lanes were dangerous or confusing.

Of the 63 police officers surveyed, 29 percent believed that there was some danger to pedestrians, and the 612 bus operators surveyed submitted a total of 393 pedestrian safety improvement suggestions. However, many of these were multiple comments from a smaller number of drivers. The people who must deal most directly with lanes on a day-by-day basis as a part of their job have noted a number of areas where safety improvements can be made. However, as noted in an earlier section, both the police and bus operators are clearly in favor of retaining the contraflow bus lanes.

Accident Analysis

The best test of the public opinion that a possible pedestrian safety hazard exists is an analysis of

actual accident data along all of the streets involved. To this end, the Bureau of Traffic Engineering and Operations conducted a comprehensive evaluation of vehicular and pedestrian accidents on all of the east-west streets that carried buses, both before and after the consolidation of the bus routes into the four contraflow bus lanes.

The accident analysis of the contraflow bus lanes considered the downtown east-west streets as two groups. Van Buren, Jackson, Adams, and Monroe streets were the group of streets whose traffic characteristics were affected by the consolidation of bus routes onto the Jackson and Adams contraflow bus lanes. A before-and-after tabulation of accident data was done for these four streets. Inasmuch as almost all of the buses had been consolidated onto Jackson and Adams streets, nearly all bus-related after accidents occurred on these streets.

A second group of streets (Madison, Washington, Randolph, and Lake streets) had traffic patterns affected by the consolidation of bus routes onto the Madison-Washington contraflow bus lanes. The aforementioned before-and-after rationale was also used in the traffic accident tabulation of these streets. The accident data used in the analyses of the Adams-Jackson group of streets contained 24 months of before data and 30 months of after data. For the Madison-Washington group of streets, 36 months of before data and 18 months of after data were available. The before-and-after traffic accident data for these two groups of streets are summarized in Table 4. The data are expressed in accidents per month to simplify comparisons.

TABLE 4 Contraflow Bus Lane Summarized Traffic Accident Experience

	Accident Experience (accidents/month)		
	Before	After	Last 6 Months Alone
Combined Avg Monthly Traffic Accident Data for Van Buren, Adams, Jackson, and Monroe[a]			
All accidents	84.3	86.6	72.2
All pedestrian accidents	6.5	8.0	4.5
All bus accidents	9.6	8.3	4.0
Bus-pedestrian accidents only	0.5	2.4	1.2
Bus-vehicle accidents only	9.1	5.9	2.8
Combined Avg Monthly Traffic Accident Data for Madison, Washington, Randolph, and Lake[b]			
All accidents	93.5	83.5	81.2
All pedestrian accidents	8.9	10.7	8.0
All bus accidents	12.9	10.7	6.9
Bus-pedestrian accidents only	0.8	2.7	2.2
Bus-vehicle accidents only	12.1	8.0	4.7

[a]Based on 24-month before and 30-month after data.
[b]Based on 36-month before and 18-month after data.

The types of accidents chosen for analysis, in addition to the total number of accidents on the eight streets involved, were total pedestrian accidents, all accidents involving CTA buses, only those pedestrian accidents reported as occurring with CTA buses, and only those vehicular accidents reported as occurring with CTA buses. In the last three categories all CTA bus accidents were included whether or not the buses were traveling in the contraflow lanes.

Analysis of these data indicates that there has been an increase in bus-pedestrian accidents on downtown streets. On the four streets affected by the Adams-Jackson bus lanes, an initial jump in bus-pedestrian accidents has been followed by a decline that has left the average rate at only 0.7 bus-pedestrian accidents per month higher than before the installation of the bus lane. On Madison-Washington bus lanes the rate is up by 1.4 bus-pedestrian accidents per month. However, the increase in this type of accident has been offset by a substantial reduction in total bus accidents (52 percent) and all pedestrian accidents (19 percent) on both sets of streets. The total of all accidents has decreased 10 percent in the last 6 months, as compared with the before condition.

To further describe the nature of the pedestrian safety experience on these streets, two graphs were prepared, which break the total pedestrian and bus-pedestrian accident data down into 3-month groupings for each set of streets (Figures 2 and 3). These graphs demonstrate just how variable these types of accidents are, with peaks and valleys occurring both before and after the installation of the contraflow bus lanes. They also both show a definite peak immediately after the lane installation, and a general decline after those dates.

Figure 4 is a spot map that shows all the bus-pedestrian accidents that have occurred on the contraflow bus lanes since the start of their operation. This map shows a high concentration of bus-pedestrian accidents on Adams Street at the entrance to Union Station just west of the river, and on three of the four contraflow bus lane streets (Washington, Adams, and Jackson) as they cross the State Street transit mall. This map identifies those areas where additional corrective action is most needed and would have the most beneficial impact.

The Bureau of Traffic Engineering and Operations, in conjunction with CTA, also conducted a video camera study of bus-pedestrian interactions along the contraflow bus lanes. This video study also documented violations of the bus lanes by other motor vehicles.

These video studies were largely concentrated at the specific problem areas previously identified. These video displays indicate that many pedestrians cross the street at all locations, in and out of the crosswalks, and they do not look in either direction as they cross a street. On streets where parking occurs this is not too great a problem, because parked cars protect the pedestrian for the first 7 or 8 ft of their journey, giving motorists and bus drivers an opportunity to warn them or take evasive action. However, whenever vehicles are traveling in the lane immediately adjacent to the curb, and the pedestrian suddenly decides to cross the street, there is little time for the driver to take appropriate countermeasures. This suggests that any improvements to signing and markings that might help further alert the pedestrian to this unusual situation would be beneficial; also, physical barriers (such as pedestrian fencing) to force pedestrians to cross at the expected crosswalk locations could make a significant difference.

Contraflow Bus Lane Experience in Other Cities

An evaluation of the experience of other cities with contraflow bus lanes has revealed that the bus lanes have been generally considered successful. However, most operations are not the same as that in Chicago, either in that the lanes are wider or that they are not located in CBDs. The one installation that is similar to Chicago's is in Pittsburgh. City officials there believe that these bus lanes are successful, although they noted that some of the public considers these bus lanes to be a safety problem. City officials do not believe this to be the case,

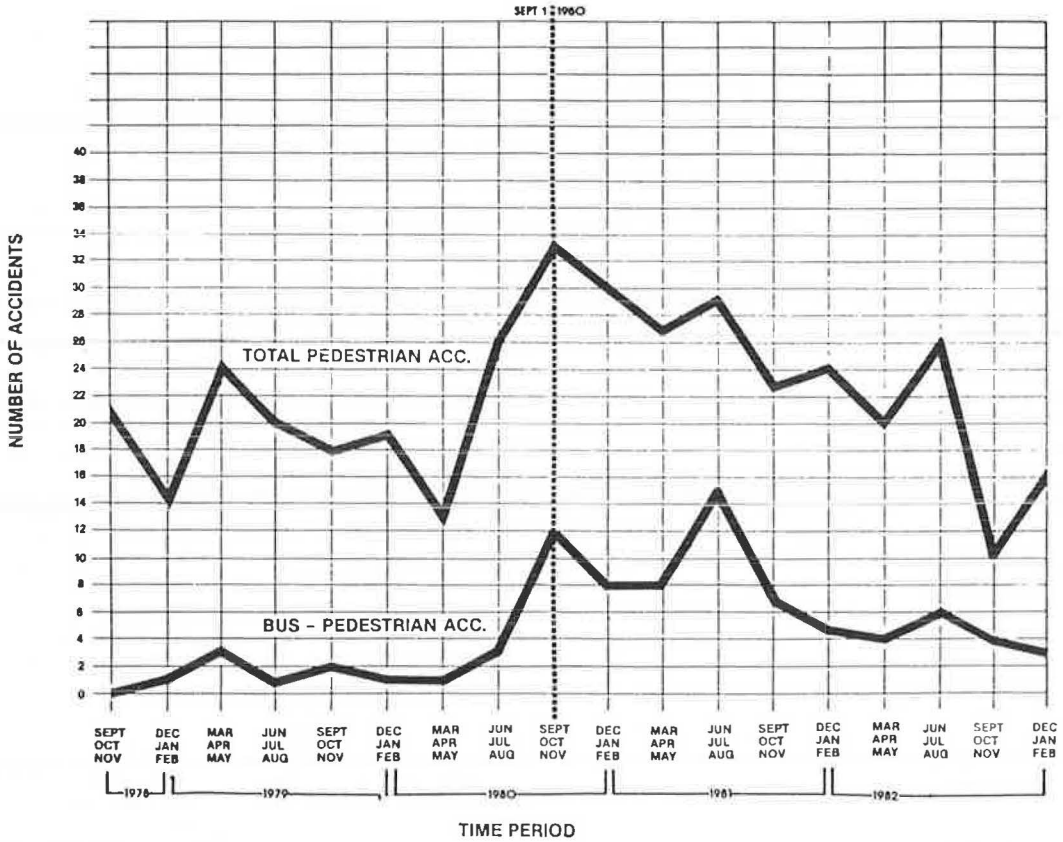

FIGURE 2 Downtown contraflow bus lane traffic accident statistics for Van Buren, Jackson, Adams, and Monroe.

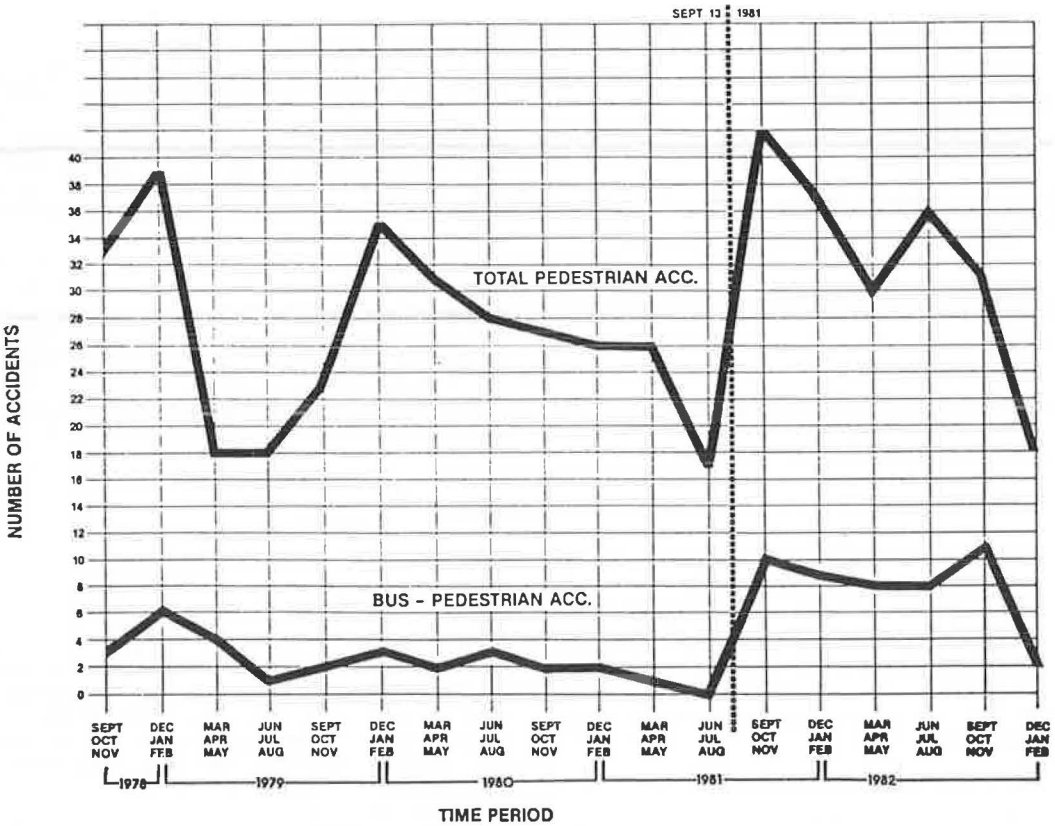

FIGURE 3 Downtown contraflow bus lane traffic accident statistics for Madison, Washington, Randolph, and Lake.

FIGURE 4 Bus-pedestrian accidents.

however, and no documentation regarding this operation has been prepared.

RECOMMENDATIONS

The primary conclusion of this study is that the bus lanes are operating in a relatively safe and effective manner, and it is recommended that they be retained. There are some operational and safety problems, but these problems are manageable and are already being addressed through various actions of the Mayor's Traffic Management Task Force.

Following is a list of specific recommendations for improvement, along with the actions already taken in each area.

Improved Signing and Pavement Markings

Less Confusing Signing

In response to numerous suggestions from both citizens and the police, new one-way signs have been posted at all of the cross streets. The sign previously used consisted of a standard one-way arrow sign with the message EXCEPT CTA BUSES at the bottom. The new signs were installed in late April 1983 and use the standard black-on-white one-way arrow sign followed by a black-on-yellow arrow sign with the work BUSES (see Figure 5).

State Street Mall Signing

There are no one-way signs posted at the State Street transit mall because the only vehicles on the mall are buses and emergency vehicles that can turn

FIGURE 5 One-way sign modification.

in either direction on the bus lane streets. New pedestrian-oriented signs that show two or three black-on-white arrows in one direction, and a black-on-yellow arrow with the word BUSES in the other direction, were posted facing all four crosswalk movements on State Street in June 1983 (see Figure 6).

FIGURE 6 One-way signing at State Street.

More Frequent Pavement Markings

The Department of Streets and Sanitation has been asked to apply the pavement markings on the contraflow bus lane streets twice a year instead of the usual once yearly CBD pavement marking. This schedule will ensure that these markings are fresh and visible throughout the year. In addition, the method of separating the opposing traffic directions has been changed from yellow cross-hatching to two 8-in. parallel yellow lines, which are much easier and cheaper to apply, thus reducing the time and cost of this operation.

Curb Line Pavement Markings

The addition of an 8-in. yellow line on top of the curb adjacent to the contraflow bus lanes should further help to alert pedestrians to the presence of buses in that curb lane. This curb marking was first installed in fall 1982.

Special Pavement Messages

The application of the WATCH FOR BUSES pavement marking message has been expanded to include various midblock locations where sizable pedestrian crossings have been observed, in addition to the usual crosswalk application of this message (see Figure 7).

Pedestrian Fencing

Union Station

In November 1982 special pedestrian fencing was installed along both the north and south curbs of

FIGURE 7 WATCH FOR BUSES pavement marking.

Adams Street between Canal Street and Wacker Drive to reduce the haphazard and dangerous pedestrian crossings throughout this area (see Figure 8). Although it is too early to develop any meaningful before-and-after accident statistics, video studies of this area reveal much more orderly and safe pedestrian traffic flows in this area.

State Street Transit Mall

A test installation of a more aesthetically designed pedestrian fencing is to be installed on the south curb of Adams Street east of State and on the north curb of Washington Street west of State in order to better channel pedestrians into the crosswalks and

FIGURE 8 Adams Street pedestrian fencing.

FIGURE 9 Improved pedestrian fence design.

reduce midblock crossings. This more aesthetic design was developed with the Bureau of Architecture (see Figure 9) and will be installed with the cooperation of the Department of Streets and Sanitation.

Additional Pedestrian Fencing

The Bureau of Traffic Engineering and Operations will be asking for a $40,000 budget item next year to cover the installation of additional aesthetically pleasing pedestrian fencing (as needed) or to replace some of the older standard fencing where there are complaints regarding its appearance.

Bus Operator Safety Training

Current Training Procedures

Currently, the CTA gives all bus operators special instructions on how to drive on the contraflow bus lanes. In addition, street supervisory personnel are instructed to constantly observe the bus lane operation and report any problems or questionable operating procedures, and bus operator instructors are regularly assigned to ride these buses and take whatever corrective measures may be needed.

Use of Video Tapes

The CTA is planning to use specially edited versions of the contraflow bus lane video tapes as an additional training tool for bus operators. Viewing of these tapes will give both new and regular bus operators some feeling of the unexpected pedestrian movements that regularly take place along these lanes, and a much better idea of where and how these incidents would most likely occur.

Continued Surveillance

On-Site Inspection

The Police Department, the CTA, and the Department of Public Works will continue to monitor the operation of the contraflow bus lanes, taking whatever corrective actions are necessary as soon as a problem develops.

Ongoing Accident Analysis

The Bureau of Traffic Engineering and Operations will continue to maintain up-to-date accident records of the contraflow bus lane streets to identify both long-term trends and specific problem locations. Again, immediate action will be taken as soon as a problem becomes apparent.

Traffic Management Task Force

The Mayor's Traffic Management Task Force will continue to monitor the activities of all its member agencies with respect to the contraflow bus lanes, act as a sounding board for complaints and problems brought to it from outside sources, and make whatever regular reports are necessary to maintain a continued public awareness and appreciation for the contraflow bus lane operation.

ACKNOWLEDGMENT

The authors wish to acknowledge the assistance provided by the rest of the Mayor's Traffic Management Task Force in the gathering of data and the preparation of this report. Specifically, the authors acknowledge the work done by Joseph Ligas and his colleagues at the Chicago Area Transportation Study in conducting the attitudinal surveys, Harold Hirsch and Frank Barker in their work with CTA bus drivers, Captains James Connelly and David Coffey in their assistance in obtaining Police Department cooperation and data, Chester Kropidlowski for his traffic safety insights and data compilation, Pat Woodburn for her invaluable efforts as recording secretary of the Mayor's Traffic Management Task Force, director David Schulz of the Budget Office and deputy superintendent Marilynn O'Regan for their consecutive chairmanships of the Mayor's Traffic Management Task Force during this period, and, most important, Kay Cafferata for her indefatigable efforts in assembling and typing these reports.

REFERENCE

1. Contraflow Bus Lanes Survey Results. Chicago Area Transportation Study, Chicago, Dec. 1982.

Publication of this paper sponsored by Committee on Transportation System Management.

Traffic Restraint on New York City's East River Bridges

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ABSTRACT

The analysis of traffic impacts associated with the New York City Department of Transportation's 1980 proposed morning peak-period driver-only ban on the four East River bridges is summarized. The ban would involve some 25,000 out of the 94,000 vehicles that enter Manhattan from 6:00 to 10:00 a.m. on the four free and three toll East River crossings. Its goal was to manage capacity consistent with transportation, economic, and air quality objectives. Driver-only cars occupy about half of all Manhattan-bound road space between 8:00 and 9:00 a.m., yet they carry less than 25 percent of the people. The traffic impact analysis considered likely changes in where, when, and how people travel. The changes were based on the equilibrium condition that would occur as traffic continually redistributes to where there is capacity. The analysis indicated that about 65 percent of the 25,000 driver-only cars on the free bridges would be diverted to toll crossings. The remaining 35 percent would be distributed in a variety of other ways. Under equilibrium conditions it is expected that queues would dissipate by 10:30 a.m. on both the Midtown Tunnel and Brooklyn-Battery Tunnel (currently, queues last until about 9:00 a.m. on the Brooklyn-Battery Tunnel and 10:00 a.m. on the Midtown Tunnel). These estimates assume that the reversible lanes would be available on both of these facilities by 6:00 a.m. A contraflow bus lane on the approach to the Brooklyn-Battery Tunnel was implemented during 1980 as a traffic management complement. However, the ban was not allowed by the state court. The community and court response suggests that implementing such automobile-restraint measures will be a difficult task in U.S. cities.

The procedures used in analyzing the traffic impacts associated with the New York City Department of Transportation's (DOT) 1980 proposal to ban driver-only cars from the four free East River crossings during the weekday morning rush periods are described. Also, the associated planning and policy implications are summarized.

The proposal to ban driver-only cars was set forth by New York City DOT in June 1980. This demonstration project was suggested as a response to the New York City DOT's desire to reduce car trips in Manhattan. It was proposed for implementation by October 1980. Adding a toll to the free East River crossings--a much discussed proposal--was ruled out because of the time, costs, and impacts involved. The analyses herein reflect both the city's policy and the time constraints that were placed on the analysis.

CONCEPT

The number of vehicles entering Manhattan has nearly

doubled during the past 30 years. This trend is continuing at a rate of approximately 1.5 percent per year. This large increase has strained Manhattan's ability to handle automobile traffic, and it has necessitated a reevaluation of New York City's transportation philosophy. In the past, efforts were directed toward accommodating all vehicles that chose to enter Manhattan. With the continuing increase in traffic volumes, it has become apparent that the limited street space must be managed more effectively. To this end a hierarchy of vehicular trips was established, ranking driver-only cars [single-occupant vehicles (SOVs)] behind such uses as transit, taxis, commercial vehicles, and multi-occupant automobiles.

The proposed ban was designed to restrict the movement of single-occupant automobiles into Manhattan across the East River on the four city-owned free bridges. Drivers without passengers would not be allowed to enter Manhattan from 6:00 to 10:00 a.m., Monday through Friday, on the Queensboro, Williamsburg, Manhattan, and Brooklyn bridges (Figure 1). Under the plan cars with passengers, handicapped drivers, buses, taxis, trucks, and emergency vehicles would be exempt from the ban and would be allowed to use the bridges during these periods.

All types of vehicles would be permitted to use the three toll facilities operated by the Triborough Bridge and Tunnel Authority (TBTA)--Triborough Bridge, Queens-Midtown Tunnel, and Brooklyn-Battery Tunnel.

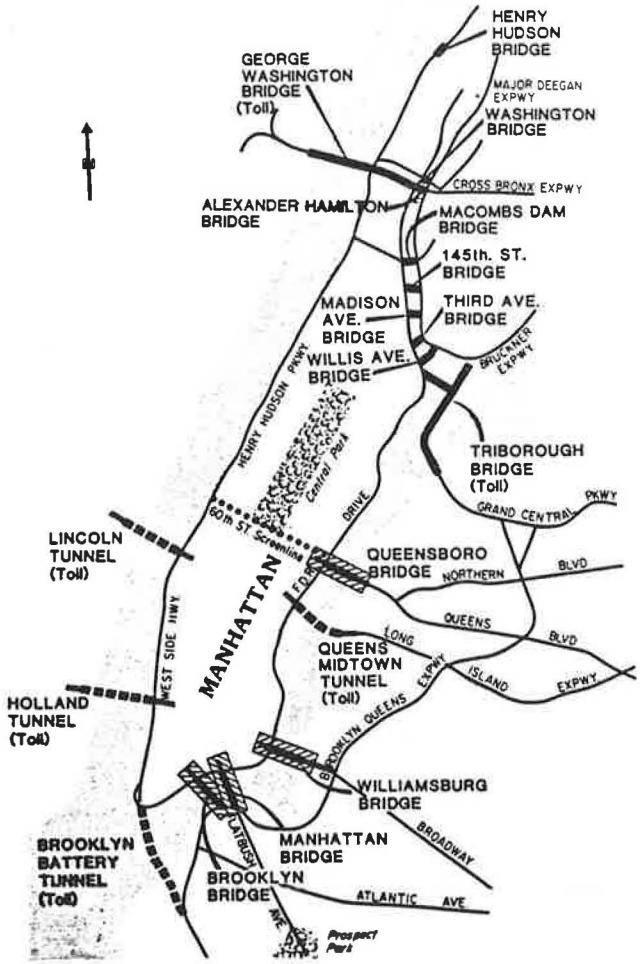

FIGURE 1 Proposed driver-only car ban, East River bridges, 6:00-10:00 a.m.

REASONS FOR THE PLAN

The goal of this plan is to manage the vehicular capacity of the East River crossings in a manner that is consistent with transportation, economic, and air quality objectives of New York City. The plan was structured to give preference to person capacity (the movement of people) rather than to vehicles. It was, perhaps, the boldest and most innovative traffic-restraint proposal for a major metropolitan area in the United States. In a sense, it adapted the Singapore automobile licensing and restraint concept to New York City (1).

In theory the driver-only car ban would apply road pricing (toll) and congestion pricing (travel time) to require peak-period driver-only cars to more equitably pay their share of transportation and nontransportation costs. The high cost of peak-hour automobile travel into congested city centers (such as Manhattan) has been recognized for several decades by transportation planners and economists.

The proposal attempts to redress the allocation of street space between driver-only cars and other vehicles. Driver-only cars occupy about half of all Manhattan-bound road space on the bridges from 8:00 to 9:00 a.m., but they carry less than 25 percent of the people. Finally, the high dependence on public transit for journeys to and from Manhattan (up to 90 percent of all person trips in peak periods) is recognized in the proposal.

The demonstration project was designed to provide a real-world assessment to the following questions.

1. Who will it help and who will it affect? Where will impacts take place?
2. How will toll revenues be affected?
3. How many people will change travel modes or paths?
4. What impacts will it have on air quality and congestion over the long run?
5. How will it influence Manhattan's economy?
6. How will it be enforced to keep violations to a minimum?
7. Is it worth continuing after a 3-month demonstration project? If it is continued, should it be modified?
8. Will the additional queues at toll plazas result in a long-run change in trip times or modes?
9. What are the legal implications?

EXISTING TRAFFIC CONDITIONS

Existing traffic conditions during the morning rush period were analyzed for each of the seven East River crossings. Traffic volume, speed and delay, and capacity data obtained from public agencies were supplemented by field studies of queuing. Traffic demands were derived by adding the number of vehicles queued to the recorded traffic volumes. The driver-only cars that use each river crossing were determined by field surveys.

Daily Traffic Volumes

Daily traffic volumes across the East River are given in Table 1. Some 600,000 vehicles crossed the East River in 43 lanes on a typical 1979 weekday. The Queensboro Bridge and the Brooklyn Bridge carried the highest flows--134,000 and 91,000, respectively. The Brooklyn-Battery Tunnel was the lightest used, carrying about 60,000 vehicles per day. Some 88 percent of all vehicles were passenger cars or taxis, 11 percent were trucks, and 1 percent were buses. The Manhattan Bridge, followed by the Queens-

TABLE 1 1979 Daily Traffic Volumes Across East River Into and Out of Manhattan

Crossing	Levels	Lanes	No. of Vehicles: Cars	Buses	Trucks	Total	Percent
Triborough Bridge (Manhattan)[a]	1	6	85,605[b]	250[c]	2,030[c]	87,885[b]	14.6
Queensboro Bridge	2	9	114,359	2,227	17,380	133,966	22.3
Queens-Midtown Tunnel[a]	1	4	61,669[c]	1,189[c]	6,969[c]	69,827	11.6
Williamsburg Bridge	1	8	65,663	544	15,133	81,340	13.6
Manhattan Bridge	2	6	53,225	137	22,041	75,403	12.6
Brooklyn Bridge	1	6	90,272	64	983	91,319	15.2
Brooklyn-Battery Tunnel[a]	1	4	56,213	1,393[c]	2,839[c]	60,445	10.1
Total[d]		43	527,006	5,804	67,375	600,185	

Note: Data are from New York City DOT.
[a]Toll facility with a $0.75 charge.
[b]Preliminary data.
[c]Estimated by New York City DOT.
[d]Total vehicle percentages are as follows: cars = 87.8 percent, buses = 1.0 percent, and trucks = 11.2 percent.

boro and Williamsburg bridges, carried the greatest number of trucks, with 22,000, 17,400, and 15,100 trucks, respectively. The Queensboro Bridge carried the largest number of buses on a daily basis, although the bus flows through the Queens-Midtown and Brooklyn-Battery tunnels are heavier in the peak hours.

Peak-Period Volumes

Westbound peak-period traffic flows on the seven river crossings are given in Table 2 (2). Some 94,000 vehicles entered Manhattan during the 4-hr period, of which 27,400 entered between 7:00 and 8:00 a.m. and 29,000 between 8:00 and 9:00 a.m. The maximum volumes carried in a single hour were as follows:

1. Queensboro Bridge, 7:00 to 8:00 a.m.--5,690 vehicles (five lanes),
2. Triborough Bridge, 8:00 to 9:00 a.m.--3,860 vehicles (three lanes) (Manhattan Plaza),
3. Brooklyn Bridge, 8:00 to 9:00 a.m.--3,780 vehicles (three lanes),
4. Brooklyn-Battery Tunnel, 8:00 to 9:00 a.m.--3,750 vehicles (three lanes),

TABLE 2 Hourly Variations in Westbound Traffic on East River Crossings *(2)*

	Triborough Bridge	Queensboro Bridge	Queens-Midtown Tunnel	Williamsburg Bridge	Manhattan Bridge	Brooklyn Bridge	Brooklyn-Battery Tunnel	Total, All Crossings
6:00-7:00 a.m.								
Driver-only cars	1,025	1,958	783	1,372	1,100	1,307	1,091	8,636
Other cars	684	1,305	522	914	734	871	728	5,758
Taxis	70	350	60	30	50	100	40	700
Buses	10	29	60	18	0	0	40	157
Trucks	80	423	250	302	344	4	60	1,463
Total vehicles	1,869	4,065	1,675	2,636	2,228	2,282	1,959	16,714
Total passenger car units (PCUs)	1,914	4,291	1,830	2,796	2,400	2,284	2,009	17,524
7:00-8:00 a.m.								
Driver-only cars	2,140	2,328	1,538	1,511	1,496	1,931	1,841	12,785
Other cars	1,427	1,551	1,026	1,008	997	1,288	1,227	8,524
Taxis	120	734	83	40	62	145	80	1,264
Buses	20	46	80	41	8	0	75	270
Trucks	80	1,032	359	663	733	14	100	2,981
Total vehicles	3,787	5,691	3,086	3,263	3,296	3,378	3,323	25,824
Total PCUs	3,837	6,230	3,305	3,616	3,667	3,385	3,410	27,450
8:00-9:00 a.m.								
Driver-only cars	2,159	2,279	1,605	1,219	1,119	2,124	2,048	12,553
Other cars	1,440	1,519	1,070	812	747	1,416	1,365	8,369
Taxis	140	577	167	63	161	204	90	1,402
Buses	30	50	160	7	0	0	150	397
Trucks	90	1,085	351	1,100	1,537	32	100	4,245
Total vehicles	3,859	5,460	3,353	3,201	3,564	3,776	3,753	26,966
Total PCUs	3,919	6,003	3,609	3,754	4,332	3,792	3,878	29,287
9:00-10:00 a.m.								
Driver-only cars	1,952	2,025	1,627	1,075	1,045	1,584	1,856	11,164
Other cars	1,301	1,349	1,084	717	697	1,056	1,237	7,441
Taxis	220	442	249	40	80	114	100	1,253
Buses	20	62	114	10	1	2	40	249
Trucks	90	1,044	423	1,048	1,224	31	90	3,950
Total vehicles	3,583	4,922	3,497	2,890	3,055	2,787	3,323	24,057
Total PCUs	3,638	5,475	3,765	3,419	3,668	2,803	3,388	26,156
Total—6:00-10:00 a.m.								
Driver-only cars	7,276	8,590	5,553	5,177	4,760	6,946	6,836	45,138
Other cars	4,852	5,724	3,702	3,451	3,175	4,631	4,557	30,092
Taxis	550	2,103	559	173	361	563	310	4,619
Buses	80	187	414	76	9	2	305	1,073
Trucks	340	3,534	1,383	3,113	3,838	81	350	12,639
Total vehicles	13,098	20,138	11,611	11,990	12,143	12,223	12,538	93,561
Total PCUs	13,308	21,999	12,509	13,585	14,607	12,264	12,685	100,417

Note: Each bus and truck represents 1.5 PCUs. Driver-only cars = 60 percent of total cars. The number of westbound lanes is as follows: Triborough Bridge = 3, Queensboro Bridge = 5, Queens-Midtown Tunnel = 3, Williamsburg Bridge = 4, Manhattan Bridge = 3, Brooklyn Bridge = 3, and Brooklyn-Battery Tunnel = 3, for a total of 24 westbound lanes.

5. Manhattan Bridge, 8:00 to 9:00 a.m.--3,560 vehicles (three lanes),
6. Queens-Midtown Tunnel, 9:00 to 10:00 a.m.--3,500 vehicles (three lanes), and
7. Williamsburg Bridge, 7:00 to 8:00 a.m.--3,276 vehicles (three lanes).

In general, peak-hour use of each bridge reflects the available roadway capacity.

Driver-Only Cars

The New York City DOT estimated that driver-only cars accounted for 60 percent of all westbound passenger vehicles during the 4-hr morning peak period. The 94,000 westbound vehicles on the seven crossings between 6:00 and 10:00 a.m. included

- 25,600 driver-only cars on the free bridges,
- 31,100 other vehicles on the free bridges, and
- 37,300 vehicles on the toll crossings (driver-only cars, cars with passengers, and trucks).

Driver-only vehicles accounted for 48 percent of all entering vehicles but transported only 25 percent of the people entering across the East River. In contrast, buses represented 1 percent of the vehicles but carried 24 percent of the people. Thus SOVs carried a disproportionately small share of the total passenger load relative to the street space they occupied.

Initial Capacity Estimates

Passenger car units (PCUs) were computed for the peak hours on each facility by assuming that each bus or truck (including light trucks) was equivalent to 1.5 passenger cars (results are given in Table 2). The resulting PCUs for the 8:00 to 9:00 a.m. period are compared with estimated capacities (service level E) in Table 3. Independent capacity estimates from an earlier source are given for comparative purposes. The capacities coordinate closely with actual peak-hour volumes, which indicate that the river crossings essentially operate at capacity during this hour.

Queuing Analysis

Field checks of existing congestion on approaches to the East River crossings are shown in Figure 2. Congestion develops on approaches to all river crossings, which indicates that there is little if any capacity reserve in the system. The congestion that builds up in advance of the Queens-Midtown Tunnel and the Brooklyn-Battery Tunnel are especially significant because these queues may be extended when the driver-only ban is enacted.

Figure 3 shows the queues on approaches to the tunnel crossings by time of day and length of queue as observed during mid-1980. This diagram quantifies the vehicle hours of delay associated with the existing facilities.

Refined Volume-Capacity Analysis

Existing queues were related to flows on the Queens-Midtown and Brooklyn-Battery tunnels to identify actual demands and to refine capacity estimates. It was assumed that the recorded traffic flow represents the actual capacity wherever queues exist. The demand is the sum of the capacity and the observed queue during the same time period. During the 4-hr period demand equaled the westbound volume, exceeded it during the height of the peak, and fell short at other times.

The data in Tables 4 and 5 give the details of the volume-capacity queuing analysis. Salient results are as follows.

1. Maximum queues on approaches for the Midtown Tunnel approximate 625 vehicles. Queues dissipate by or before 10:00 a.m.
2. Maximum queues on approaches to the Brooklyn-Battery Tunnel approximate 170 vehicles. Queues dissipate by 9:00 a.m.

The data in Tables 6 and 7 indicate how these queues would be reduced by opening toll booths earlier and by initiating lane reversals by 6:00 a.m. on the Brooklyn-Battery Tunnel and by 6:30 a.m. on the Queens-Midtown Tunnel. With earlier lane reversals, the maximum queues on the Midtown Tunnel would dissipate by 9:15 a.m. The lane reversals on the Brooklyn-Battery Tunnel would virtually eliminate all queuing on the approaches.

The demands, capacities, and queues given in these tables were used in deriving actual traffic impacts of the driver-only ban on the four free bridges.

AFFECTED MOTORISTS

The proposed driver-only ban would affect some 25,000 motorists on the four free East River bridges

TABLE 3 Volume-Capacity Comparisons Across the East River, 8:00 to 9:00 a.m. Westbound

					Capacity (PCUs)	
Crossing	Lanes, 1980	Driver-Only Cars	Other Vehicles	Total PCUs	Creighton[a]	Preliminary Capacity Estimate
Triborough Bridge	3	2,159	1,760	3,919	4,080	4,080
Queensboro Bridge	5	2,279	3,723	6,002	7,450[b]	6,000[c]
Queens-Midtown Tunnel	3	1,605	2,004	3,609	4,170	4,000
Williamsburg Bridge	4	1,219	2,535	3,754	3,800	3,800
Manhattan Bridge	3[d]	1,119	3,214	4,333	4,810	4,500
Brooklyn Bridge	3	2,124	1,668	3,792	4,700	4,500
Brooklyn-Battery Tunnel	3	2,048	1,830	3,878	4,170	4,000
Total[e]	24	12,553	16,734	29,287	33,180	30,880

Note: Data are from the New York City DOT. It is assumed in the data that driver-only cars are 60 percent of all passenger cars.
[a] Data are from estimates done by Rodger Creighton and Associates.
[b] Six lanes.
[c] Five lanes.
[d] Three lanes designated, but four lanes used.
[e] Total vehicle percentages are as follows: driver-only cars = 42.9 percent, and other vehicles = 57.1 percent.

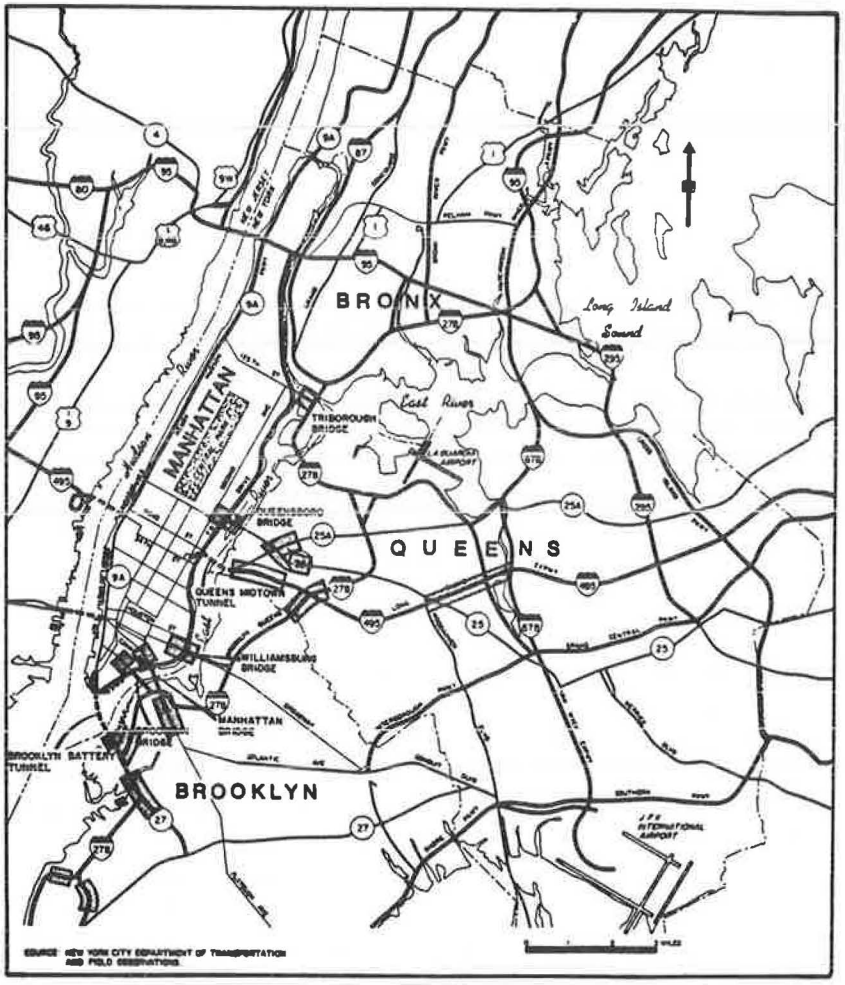

FIGURE 2 Existing queues, East River crossings, 7:30-8:30 a.m.

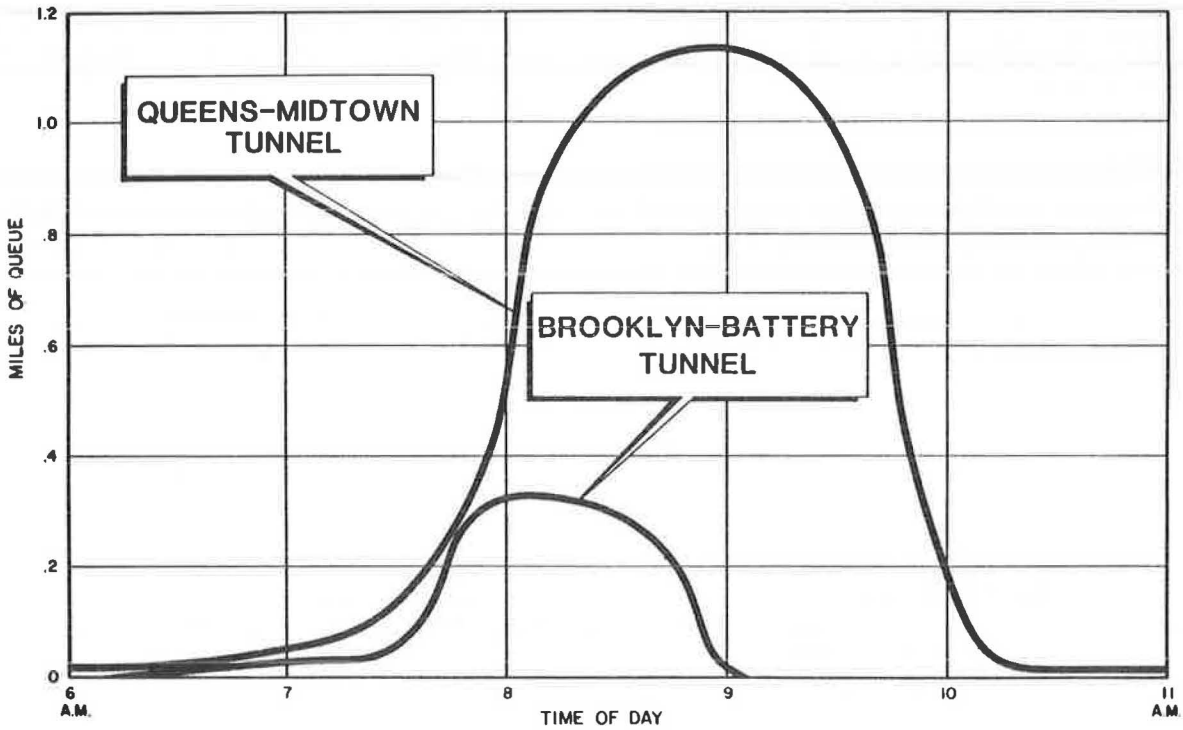

FIGURE 3 Variations in queues.

between 6:00 and 10:00 a.m. These motorists account for less than 1 percent of the 2.87 million people who enter Manhattan each day and 1.7 percent of the people who enter Manhattan from 6:00 to 10:00 a.m. These motorists account for 4.0 percent of the total daily vehicles that enter Manhattan and 27 percent of the peak-period vehicles that cross the East River from 6:00 to 10:00 a.m. The following list gives some statistics on the affected motorists [note that some data are from the Tri-State Regional Planning Commission (3,4), and the data for the driver-only cars are estimated from 1979 bridge data furnished by the New York City DOT]:

1. Total people entering Manhattan (1978): all day = 2,870,000, and between 6:00 and 10:00 a.m. = 1,504,000;

2. Total people entering Manhattan each day from Brooklyn-Queens sector (1978) = 1,364,000;

3. Number of people entering in driver-only cars on four East River bridges (1979) = 25,000; percentage of total people entering Manhattan all day = 0.87, percentage of total people entering Manhattan between 6:00 and 10:00 a.m. = 1.67, and percentage of total people entering Manhattan from Brooklyn-Queens sector all day = 1.83;

TABLE 4 Existing Demand-Capacity Queues, Queens-Midtown Tunnel

Time Beginning	1979 Westbound Volume[a]	Demand	Estimated Capacity[b]	Difference	Cumulative Queue[c] (end of period)
5:00 a.m.	793	793	2,600	1,807[d]	
6:00 a.m.	1,830	1,830	2,600	770[d]	
7:00 a.m.	1,585	1,660	1,600	60	60
7:30 a.m.	1,720	1,950	1,750	200	260
8:00 a.m.	1,800	2,085	1,750	335	595
8:30 a.m.	1,859	1,930	1,900	30	625
9:00 a.m.	1,883	1,800	1,900	100[d]	525
9:30 a.m.	1,882	1,354	1,779[e]	425[d]	100
10:00 a.m.	2,805	2,705	3,200	495[d]	
11:00 a.m.	2,540	2,540	2,900	360[d]	
12:00 noon	2,276	2,276	2,600	324[d]	
Total	20,923	20,923			

[a]Data are from New York City DOT.
[b]Reflects volumes passing through tunnel where queue exists.
[c]Field observations, 1980.
[d]Surplus.
[e]Capacity adjusted from 1,780 to 1,779 to balance volumes.

TABLE 5 Existing Demand-Capacity Queues, Brooklyn-Battery Tunnel

Time Beginning	1979 Westbound Volume[a]	Demand	Estimated Capacity[b]	Difference	Cumulative Queue[c] (end of period)
5:00 a.m.	517	517	3,400	2,883[d]	
6:00 a.m.	2,009	2,009	3,400	1,391[d]	
7:00 a.m.	1,700	1,705	1,700	5	5
7:30 a.m.	1,710	1,863	1,700	163	168
8:00 a.m.	1,950	1,940	1,950	10[d]	158
8:30 a.m.	1,928	1,780	1,950	170[d]	-
9:00 a.m.	1,710	1,710	1,950		
9:30 a.m.	1,678	1,678	1,950		
10:00 a.m.	2,022	2,022	3,400		
11:00 a.m.	1,810	1,810	3,400		
12:00 noon	1,353	1,353	3,400		
Total	18,387	18,387			

[a]Data are from New York City DOT.
[b]Reflects volumes passing through tunnel where queue exists.
[c]Field observations, 1980.
[d]Surplus.

TABLE 6 Anticipated Demand-Capacity Queues, Queens-Midtown Tunnel

Time Beginning	1979 Westbound Volume[a]	Demand	Estimated Capacity[b]	Difference	Cumulative Queue[c] (end of period)
5:00 a.m.	793	793	2,600	1,807[d]	
6:00 a.m.	1,830	1,830	3,600	1,770[d]	
7:00 a.m.	1,585	1,660	1,900	240[d]	
7:30 a.m.	1,720	1,950	1,900	50	50
8:00 a.m.	1,750	2,085	1,900	185	235
8:30 a.m.	1,859	1,930	1,900	30	265
9:00 a.m.	1,883	1,800	1,900	100[d]	165
9:30 a.m.	1,882	1,354	1,900	456[d]	
10:00 a.m.	2,805	2,705	3,800		
11:00 a.m.	2,540	2,540	3,800		
12:00 noon	2,276	2,276	3,200		
Total	20,923	20,923			

[a]Data are from New York City DOT.
[b]Assumes maximum observed flow rate through toll station.
[c]Field observations, 1980.
[d]Surplus.

TABLE 7 Anticipated Demand-Capacity Queues, Brooklyn-Battery Tunnel (existing traffic and improved operation)

Time Beginning	1979 Westbound Volume[a]	Demand	Estimated Capacity[b]	Difference
5:00 a.m.	517	517	3,400	2,883[c]
6:00 a.m.	2,009	2,009	3,900	1,391[c]
7:00 a.m.	1,700	1,705	1,950	245[c]
7:30 a.m.	1,710	1,863	1,950	87[c]
8:00 a.m.	1,950	1,940	1,950	10[c]
8:30 a.m.	1,928	1,780	1,950	170[c]
9:00 a.m.	1,710	1,710	1,950	
9:30 a.m.	1,678	1,678	1,950	
10:00 a.m.	2,022	2,022	3,900	
11:00 a.m.	1,810	1,810	3,900	
12:00 noon	1,353	1,353	3,400	
Total	18,387	18,387		

[a]Data are from New York City DOT.
[b]Assumes maximum observed flow rate through toll station.
[c]Surplus.

4. Total vehicles entering Manhattan each day (1978) = 649,000; and
5. Number of driver-only cars entering Manhattan on four East River bridges between 6:00 and 10:00 a.m. (1979) = 25,000; percentage of total vehicles entering all day = 3.85.

TRAVEL IMPACTS

Estimating impacts was a challenging procedure because there were few, if any, real precedents. Moreover, a complex series of choices are associated with the proposed ban. There would be changes in where, when, and how people travel. There would be changes in mode and route. Like any transport change, a chain reaction of impacts would occur when the project is implemented.

1. The first-day impacts would involve major shifts to toll crossings (where driver-only cars would pay the $1.00 toll). This would substantially increase the existing congestion at toll plazas.
2. Over time a new equilibrium condition would be reached as traffic continuously redistributes to where roadway capacity is available. This is a reasonable assumption in a large metropolitan area where motorists have many ways to travel from where they live to where they work.

The general impact sequence is shown in Figure 4. Driver-only cars initially would be required to shift to toll crossings, thus causing an increase in congestion levels. The increased congestion on toll crossings would cause multi-occupant vehicles to shift to free crossings and some driver-only cars to shift to alternative modes, thereby causing some people to become carpool passengers or transit riders. This would serve to reduce congestion on the toll crossings and in the long run return some traffic to the free facilities.

Over time the cyclical effect will stabilize and a net diversion will occur (Figure 5). It was assumed that this stabilization would be achieved within a 90-day test period. Thus congestion that results from implementation of the ban will be less after a period of stabilization has been reached. If such a redistribution did not take place, serious congestion would remain on the toll crossings. This would then lead to revising or discarding the demonstration.

Diversion Estimates

Estimates of equilibrium impacts were derived from various studies of traveler responses to price increases in the New York City area. Results of Hudson crossing driver surveys and Midtown automobile driver surveys provided a basis for making modal diversion estimates. These findings and the resulting diversion estimates for the East River crossings are given in Table 8.

One-third of the drivers of driver-only cars were estimated to change their travel behavior: about half of these motorists would no longer drive because they would become carpool passengers, take transit, or not make the trip across the East River. Another one-third of these drivers would remain on the free bridges but would change their time (before

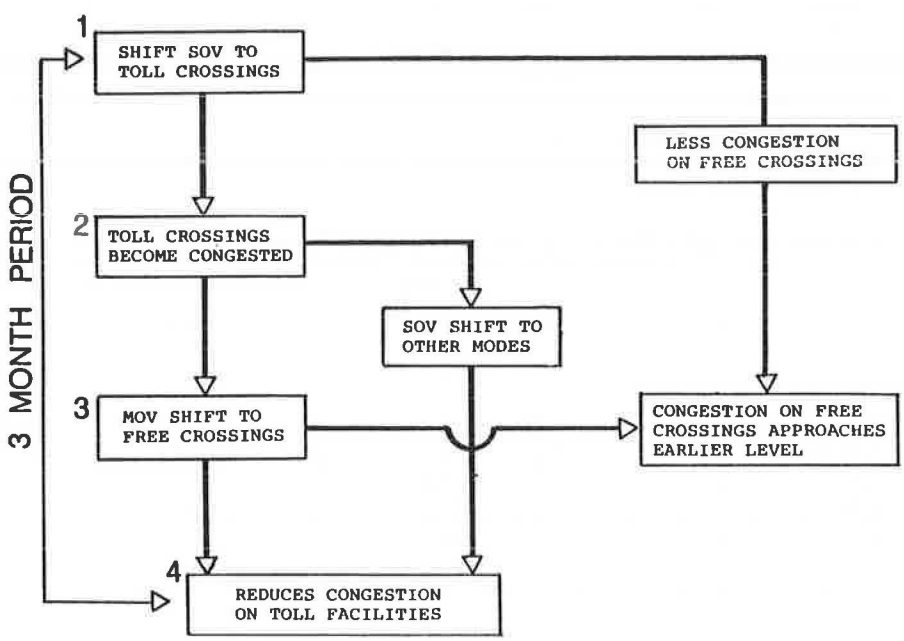

FIGURE 4 General impact sequence.

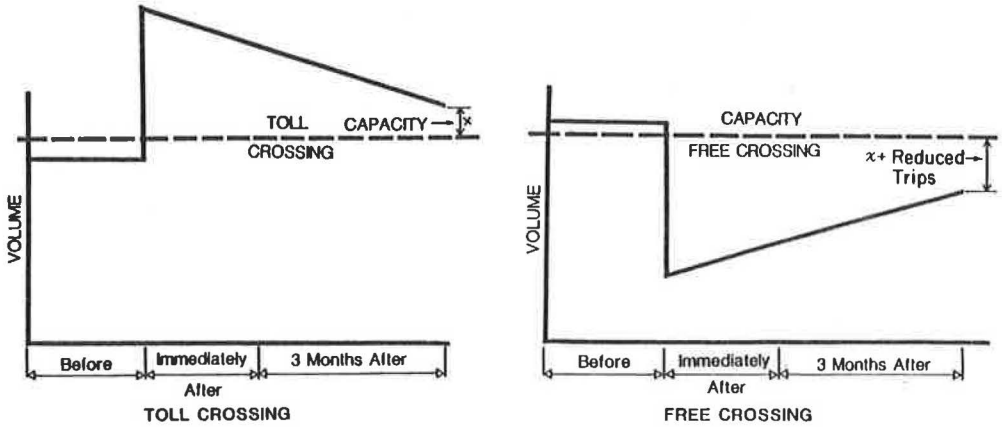

FIGURE 5 Anticipated impacts over time.

TABLE 8 Estimated Effects of a $1.00 Increase in Tolls or Parking Charges

Item	Hudson River Crossings, All Vehicles, 1978 Survey, Toll Changes[a] (%)				Midtown Automobile Driver Survey, Parking Changes[b] (%)		East River Free Crossings (%): Anticipated Traffic Reduction in Driver Only Cars (peak period)
	All Trips		Work Trips		All Trips		
	Driver Response	Traffic Reduction (peak hour)	Driver Response	Traffic Reduction (peak hour)	Driver Response	Traffic Reduction (peak hour)	
No change	73	87	77	88	70	84	67
Change	27	13	23	12	30	16	33
Join start-up carpool	4	2	4	2	-	-	11[c]
Begin taking public transit	4	4	4	4	11	11	8
Change time of day when trip began	16	5	11	3	-	-	10
Would not make trip as often (fewer trips)	2	1	2	1	11	5	2
Would not make trip at all	1	1	2	2	-	-	2
Change parking place	-	-	-	-	8	-	-

[a] Data are from Levinson et al. *(5)*.
[b] Data are from Crossley Surveys *(6)*.
[c] Assumes 5 percent carpool passengers and 6 percent new multiple-occupancy vehicles.

or after the ban), and the remainder would become carpool drivers.

Figure 6 shows details of the anticipated travel impacts of the driver-only ban. About 65 percent of the 25,000 driver-only cars on free bridges would be diverted to toll crossings, and the remaining 35 percent would be distributed as follows:

Impact	Percent	Vehicles
Stay on free bridges in peak	8	2,038
New carpools	6	
Miscellaneous vehicles	2	
Stay on free bridges off peak	10	2,547
Divert to toll crossings	65	16,557
Reduced traffic	17	4,331
Total		25,473

Quantifying Traffic Impacts

Detailed traffic assignments were made for the 25,000 diverted vehicles to the various toll crossings. The steps were as follows.

Step 1. The net traffic remaining on each of the free bridges was computed on an hour-by-hour basis by deducting the driver-only cars that would (a) represent reduced traffic (17 percent) and (b) divert to toll crossings (65 percent). In addition, the traffic that would travel outside of the peak period was reassigned: 6 percent to 10:00-11:00 a.m. and 4 percent to 5:00-6:00 a.m. These vehicles were deducted from traffic in the 4 peak hours as follows: 65 percent of the additional traffic in the 5:00 to 6:00 a.m. period was deducted from the 7:00 to 8:00 a.m. period, and 35 percent was deducted from the 8:00 to 9:00 a.m. period; similarly, 65 percent of the additional traffic in the 10:00 to 11:00 a.m. period was deducted from the 9:00 to 10:00 a.m. period, and 35 percent was deducted from the 8:00 to 9:00 a.m. period. Minor adjustments were made to reflect these time shifts in the traffic diverted to toll crossings. Some 8 percent of the driver-only cars would remain on each free crossing.

Step 2. The traffic diverted to toll facilities was reassigned to each toll crossing in accordance with traffic flow patterns based on reviews of existing travel patterns.

Step 3. Traffic impacts on toll crossings were derived as follows.

1. The diverted traffic was superimposed on the existing toll crossing demands on an hour-by-hour basis, taking into account the anticipated capacities of each toll crossing (i.e., earlier start-up and later ending of the reversible lane operations on the Brooklyn-Battery and Queens-Midtown tunnels).
2. The expected demands were compared with available capacities and queues were identified. Multiple-occupant cars on the toll crossings were

then back diverted to free bridges, based on the following two criteria: (a) 20 percent of the additional queue would remain or (b) not more than 75 percent of the eligible vehicles would shift from the toll crossings. For the Triborough and Queens-Midtown toll facilities it was assumed the 80 percent of the maximum additional queue would divert (20 percent remain). On the Brooklyn-Battery Tunnel a diversion level of 75 percent from toll to free was assumed for eligible vehicles.

3. The resulting 3-month after capacity deficiencies and queues were quantified, and adjustments were made for some shift to the 7:00 to 8:00 a.m. hour, thereby reflecting the motorists' attempt to reduce queues.

4. The resulting demands, volumes, capacity deficiencies, and queues on each toll crossing were estimated.

5. Because at 10:00 a.m. all vehicles would be eligible to use the free bridges, it was assumed that 50 percent of the vehicles queued by (or just before) 10:00 a.m. would shift to the free crossings. Based on this shift, adjusted volumes on each toll crossing were computed.

Step 4. The vehicles shifting from toll crossings to free bridges were assigned to the free bridges according to previously developed traffic distributions. This traffic was then added to the existing traffic remaining on the free bridges.

The hour-by-hour traffic volumes on the seven crossings as of October 1979 and 3 months after the proposed experiment are given in Table 9. A review of the data in this table indicates the following:

1. Total traffic from 5:00 to 11:00 a.m. would decrease from 160,800 to 156,500--about 4,300 vehicles;
2. Total traffic from 6:00 to 10:00 a.m. would decrease from about 100,400 to 90,800--about 9,600 vehicles;
3. Total traffic from 10:00 to 11:00 a.m. would increase from 19,800 to 24,100--about 4,300 vehicles;
4. There would be virtually no increase in the peak flows through the three toll plazas because of capacity restraints; and
5. The peak flows through the free bridges would decrease as follows:

Bridge	Before	After
Queensboro	6,200	5,130
Williamsburg	3,750	3,580
Manhattan	4,330	3,880
Brooklyn	3,790	3,290

FLOW AND CONGESTION IMPACTS

The traffic impacts are far broader than merely redistributing road space by type of user: (a) there would be less overall traffic during the morning peak period with net reductions in Manhattan and across the East River; (b) traffic would be reduced on city street approaches to the free bridges at Queens Plaza, Williamsburg, and downtown Brooklyn; and (c) queues would be limited to express highways that are removed from the business centers.

Current and anticipated queues on the Midtown Tunnel and Brooklyn-Battery Tunnel are given in Table 10. It is expected that queues would dissipate by 10:30 a.m. on both facilities. Maximum delays to individual vehicles could range from about 20 to 30 min (currently, queues last until about 9:00 a.m. on the Brooklyn-Battery Tunnel and 10:00 a.m. on the Midtown Tunnel). These estimates assume that the reversible lanes would be available on both of these facilities by 6:00 a.m.

The anticipated congestion impacts are shown in Figure 7. Queues would be reduced on local streets and limited to express highway approaches, where increases in their length and duration are anticipated.

The number of vehicles entering Manhattan on each toll crossing is limited by toll plaza capacity. Therefore, during most of the peak period relatively little additional traffic is expected on Manhattan streets at the exits of the three TBTA facilities.

EXPECTED BENEFITS

Estimates were also made of the impacts of the ban on vehicle miles of travel (VMT), toll revenues, transit ridership, parking revenues, and implementation costs.

VMT

The automobile driver ban is expected to result in 4,300 fewer vehicle trips each day. This corresponds to a reduction of almost 70,000 VMT daily, assuming the 8-mile vehicle trip length derived by the Tri-State Regional Planning Commission. It amounts to about 17,500,000 fewer VMT annually. The travel distances to lower Manhattan from points in Brooklyn and Queens are about the same over toll and free crossings. This is also true for trips to midtown Manhattan from the two boroughs. Therefore, it is unlikely that changes in driver trips between toll and free crossings would increase the VMT.

Toll Revenues

Almost 8,000 additional vehicles would likely use TBTA facilities to enter Manhattan each day. This corresponds to some $2,000,000 in annual revenues.

Transit Revenues

An estimated 8 percent of the 25,000 driver-only vehicles would divert to public transport services, whereas the equivalent of 1 percent would shift from transit into newly formed carpools. Some 1,750 net additional daily transit riders would generate an annual revenue of $656,000. The subway system has potential track capacity to carry some 100,000 additional riders in a single hour as compared with the 25,000 driver-only cars in a 4-hr period.

Parking

Some 150,000 vehicles park each day in the Manhattan central business district (CBD). If all 25,000 driver-only cars were removed--an unlikely condition--it would represent a 16 percent reduction. Even then some of the spaces would become available for high turnover parking, thereby reducing revenue loss to the parking industry.

A more realistic impact is the effects of the 4,300 vehicles that would no longer drive to or through Manhattan. This represents less than a 3 percent reduction in the number of parked vehicles in Manhattan at the time of the maximum parking accumulation. Assuming an average parking charge of $3.60 per day and a parking tax of 14 percent, this corresponds to a daily loss in tax revenue to the city of about $1,500, or about $375,000 per year. [These estimates assume that about 70 percent of the

TABLE 9 Current and Anticipated Westbound Traffic Volumes, Equilibrium Conditions (PCUs)

Hour Beginning	Tri-borough[a] (toll)		Queensboro[b]		Queens Midtown Tunnel[c] (toll)		Williamsburg[d]		Manhattan[e]		Brooklyn[f]		Brooklyn-Battery Tunnel[g] (toll)		All Free Crossings		All Toll Crossings		Grand Total, All Crossings	
	Before	After	Before	After	Before	After	Before	After	Before	After	Before	After	Before	After	Before	After	Before	After	Before	After
5:00 a.m.	415	415	1,684	2,028	793	793	1,430	1,637	1,025	1,215	930	1,208	517	517	5,069	6,088	1,725	1,725	6,794	7,813
6:00 a.m.	1,914	2,373	4,291	2,490	1,830	3,393	2,796	1,534	2,400	1,388	2,284	1,082	2,009	3,896	11,771	6,494	5,753	9,642	17,524	16,136
7:00 a.m.	3,837	4,080	6,230	5,126	3,305	3,800	3,616	2,739	3,667	2,703	3,385	2,196	3,410	3,900	16,898	12,764	10,552	11,780	27,450	24,544
8:00 a.m.	3,919	4,080	6,003	5,031	3,609	3,800	3,754	3,316	4,332	3,883	3,792	2,625	3,878	3,900	17,881	14,855	11,406	11,780	29,287	26,635
9:00 a.m.	3,638	4,055	5,475	4,136	3,765	3,800	3,419	2,808	3,668	3,009	2,803	1,821	3,388	3,900	15,365	11,774	10,791	11,755	26,156	23,529
Subtotal (6:00-10:00 a.m.)	13,308	14,588	21,999	16,783	12,509	14,793	13,585	10,397	14,067	10,983	12,264	7,724	12,485	15,576	61,915	45,887	38,502	44,957	100,417	90,844
10:00 a.m.	2,588	2,588	4,317	5,105	2,805	3,127	2,973	3,578	2,743	3,372	2,385	3,292	2,022	2,997	12,418	15,347	7,415	8,712	19,833	24,059
11:00 a.m.	2,357	2,357	3,764	3,764	2,540	2,540	2,601	2,601	2,707	2,707	2,182	2,182	1,810	1,810	11,254	11,254	6,707	6,707	17,961	17,961
12:00 noon	2,029	2,029	3,271	3,271	2,276	2,276	2,345	2,345	2,412	2,412	2,085	2,085	1,353	1,353	10,113	10,113	5,658	5,658	15,771	15,771
Total (5 00 a.m.-1:00 p.m.)	20,697	21,977	35,035	30,951	20,923	23,529	22,934	20,558	23,154	20,689	19,846	16,491	18,387	22,253	100,769	88,689	60,007	67,759	160,776	156,448

[a]Capacity = 4,080.
[b]Capacity = 6,000-6,300.
[c]Capacity = 3,800.
[d]Capacity = 3,800.
[e]Capacity = 4,300-4,500.
[f]Capacity = 4,700.
[g]Capacity = 3,900.

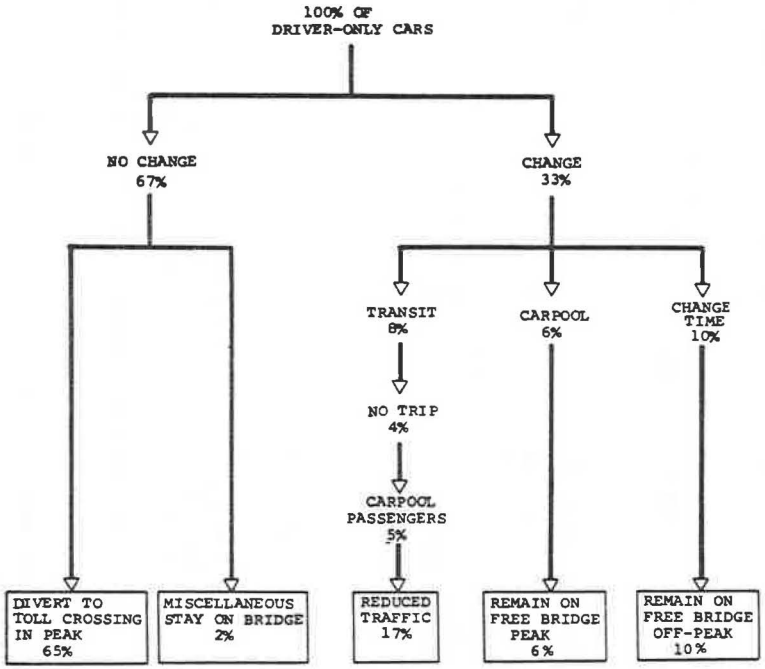

FIGURE 6 Anticipated impacts of driver-only ban on driver-only cars.

TABLE 10 Queue Characteristics

East River Crossing	Current Traffic: Existing Conditions	Current Traffic: With Improved Operations[a]	Anticipated Traffic with Improved Operations[a]: First Day (no back-diversion)	3 Months Later: Before Redistribution After 10:00 a.m.	3 Months Later: After Redistribution After 10:00 a.m.
Queens-Midtown Tunnel					
Maximum queue	625	265	3,900	1,200	1,200
Lasts until	10:00 a.m.	9:15 a.m.	1:00 p.m.	10:50-11:00 a.m.	10:15-10:30 a.m
Brooklyn-Battery Tunnel					
Maximum queue	168	-	5,700	1,950	1,950
Lasts until	9:00 a.m.	-	12:45 p.m.	11:00-11:10 a.m.	10:30 a.m.

[a]Early opening of toll booths' reversible lane.

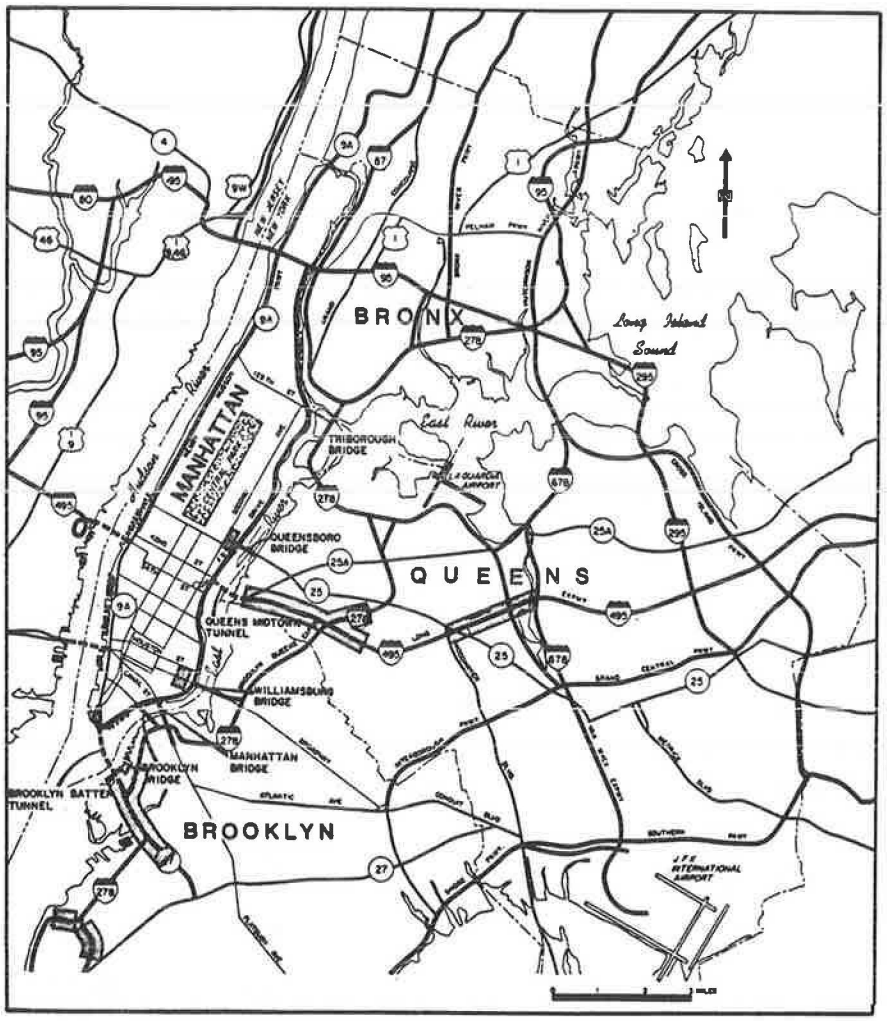

FIGURE 7 Anticipated queues with demonstration project (3 months later).

reduced trips parked in Manhattan (off street), 10 percent parked on street, and 20 percent were passing through.]

Implementation Costs

The automobile driver ban would require some 18 patrolmen to enforce it over the first 3 months, after which the number of patrolmen would be reduced to 12. The New York City DOT estimated that some 12 police officers would be required to enforce the project at an annual cost of $450,000. It was estimated by the city that minimum installation costs would be needed.

Total Costs and Benefits

In summary, the city would spend or lose $825,000 annually while the New York City Transit Authority and the TBTA would gain $2,656,000. Collectively, the public agencies would gain $1,831,000 in annual revenues.

TRAFFIC MANAGEMENT PROPOSALS

Traffic management proposals were recommended to complement the driver-only ban and to alleviate impacts of added traffic on the Queens-Midtown Tunnel (Figure 8). It was essential to initiate the reversible lane operation on the two tunnels at 6:00 a.m. to minimize queue build up. It was also necessary to provide a southbound contraflow bus lane on the Gowanus Expressway approach to the Brooklyn-Battery Tunnel to enable some 120 to 150 buses each peak hour to bypass queues.

Additional measures included the following: (a) brochures describing features of the plan and alternate routes for Queens, Brooklyn, Richmond, and Long Island motorists; (b) give advance warning notices for 2 weeks before the experiment begins; (c) extensive media publicity; (d) advance signing and trail-blazers on key approach streets and highways; (e) a TBTA campaign to encourage and increase token use (this will help speed up transactions at toll plazas); (f) emphasize park-and-ride facilities at such key locations as Shea Stadium and South Beach, and better park-and-ride use of available space at city-owned garages in Long Island City and downtown Brooklyn; and (g) adjustments in traffic signal sequences and timing on key streets and junctions (i.e., Queens Boulevard, Greenpoint Avenue, Atlantic Avenue).

STATUS AND IMPLICATIONS

The Gowanus Expressway contraflow bus lane was im-

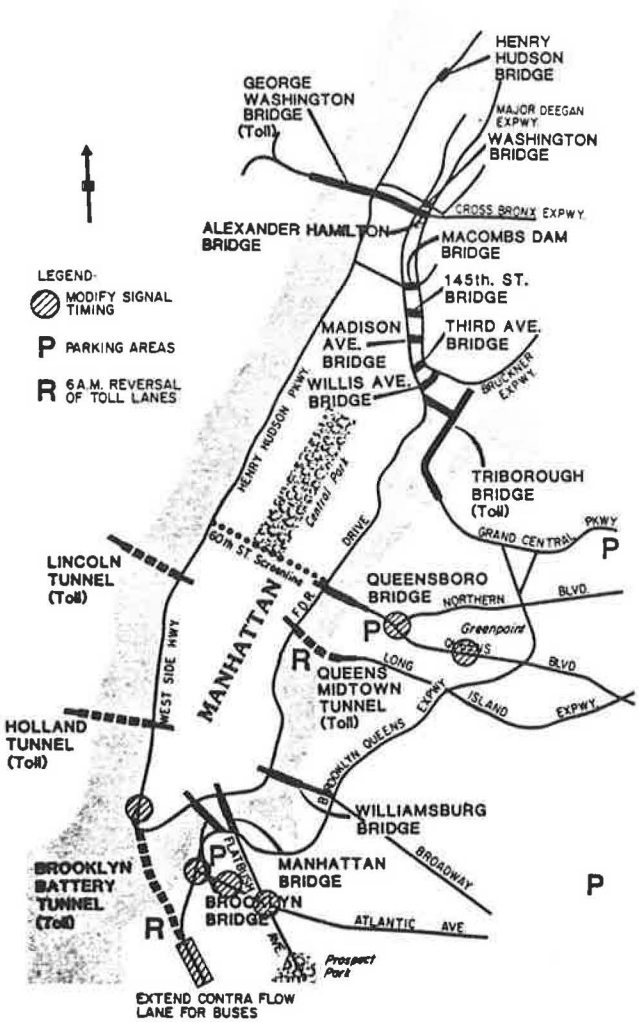

FIGURE 8 Traffic management concept.

plemented during September 1980. It saves some 5 min, on average, for more than 15,000 bus passengers each weekday during the morning peak period.

The driver-only ban was to have been implemented during September 1980 as a 3-month demonstration project. The Automobile Club of New York and the Manhattan Parking Association contested its legality in court. The court held that the city did not have the legal authority to establish such a regulation--that such legal authority rested with the state.

Restraining car use by means of an automobile driver ban has, therefore, proved difficult in New York City, the most transit-oriented central area in the United States. This implies that other U.S. cities must look carefully before they enact traffic-restraint measures.

If it were possible to place tolls on all crossings, and if there were adequate storage capacity in all toll booth plazas, it would be possible to obtain a system of equilibrium by offering positive pricing incentives at all facilities for carpools. Such a plan might resolve the legal and political difficulties, but it is not possible, at least in the short run.

The analytical approaches, however, have direct transferrability to other situations. These include use of observed queues to estimate demands, attitude surveys to estimate impacts of traffic-restraint actions, and sequential manual traffic assignments to estimate long-term impacts on various river crossings.

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