TRANSPORTATION RESEARCH RECORD 841

Freeway Operations, Railroad-Highway Grade Crossings, and Evaluating Highway Improvements

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NATIONAL RESEARCH COUNCIL

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Real-Time Freeway-to-Freeway Diversion: The San Antonio Experience

CONRAD L. DUDEK, WILLIAM R. STOCKTON, AND DONALD R. HATCHER

Studies to evaluate the effectiveness of a low-cost changeable message sign motorist-information-diversion system in San Antonio, Texas, are documented. The system was implemented as a demonstration program by the Texas State Department of Highways and Public Transportation, working in cooperation with the San Antonio Corridor Management Team. Its purpose was to alleviate congestion and reduce accidents on Interstate 35 in San Antonio near the central business district. This paper describes the system, its effectiveness, the problems encountered, and recommendations to avoid or overcome the problems. The information should be useful to others who may be implementing and evaluating a similar system.

In 1977 the Texas State Department of Highways and Public Transportation (TSDHPT), working in cooperation with the San Antonio Corridor Management Team (CMT), initiated programs aimed at alleviating congestion and reducing accidents on Interstate 35 in San Antonio near the central business district (CBD). The programs were the development, implementation, and demonstration of a low-cost motorist-information-diversion system (MIDS), which included the following phases:

1. I-35 route change around the CBD,
2. Use of a low-cost changeable message sign (CMS) system for freeway diversion, and
3. Use of the CMS system for managing traffic during freeway maintenance.

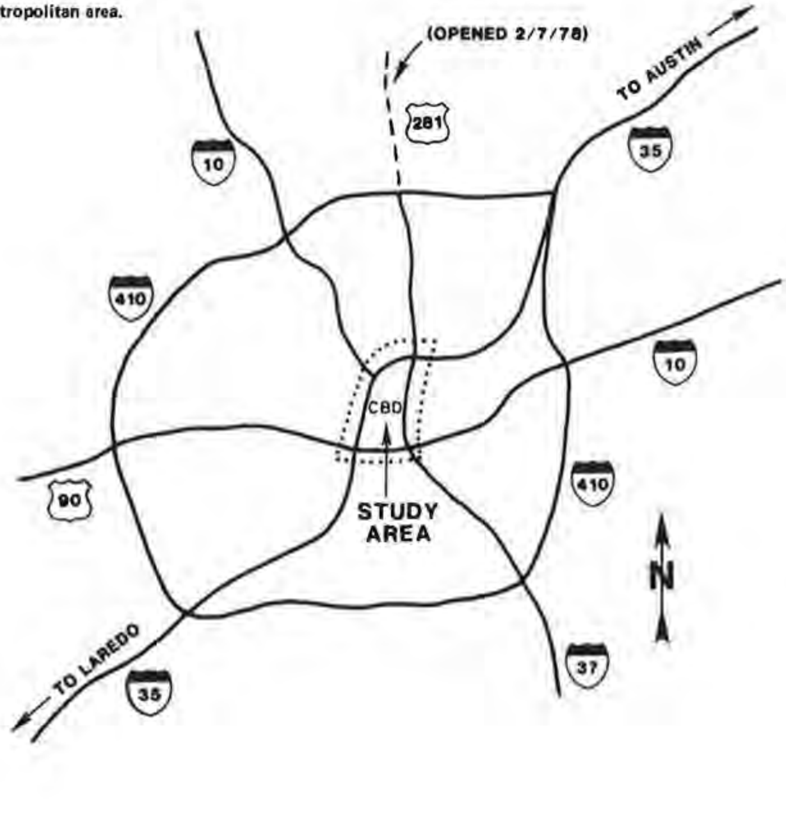
The Texas Transportation Institute (TTI) was contracted to evaluate the effectiveness of the above three programs as part of phase 2 of the Federal Highway Administration (FHWA) sponsored research entitled, Human Factors Requirements for Real-Time Motorist Information Displays. This provided an opportunity to not only evaluate the effectiveness of the specific traffic-management approaches but also to study the institutional and operational approaches used in San Antonio, and to develop hardware, operational, and evaluation guidelines for other cities in the United States that may implement and evaluate similar types of systems.

Results of the I-35 route change and the use of the CMS system for managing traffic during freeway maintenance are presented in other papers and reports (1,2). This paper discusses the use of a low-cost CMS system for freeway diversions of CBD-bound traffic.

The major freeway routes in the San Antonio metropolitan area are shown in Figure 1. I-35 is the primary facility in the Austin-Laredo corridor and is one of the oldest freeways in San Antonio. The four-lane section of I-35 that forms the north and west boundaries of the CBD was completed in 1957. Considerable congestion and relatively high accident rates are experienced on this partially elevated freeway section that has capacity restraints, such as relatively severe alignment and narrow right-of-way, particularly at the structures (3).

I-10 and I-37 are eight-lane freeways built in the late 1960s with higher design standards. As the southern and eastern boundaries, they form an alternate route around the downtown area. This route is approximately 0.8 mile (1.3 km) longer than the

Figure 1. Major highways in San Antonio metropolitan area.



primary route [5.6 miles (9.0 km) versus 4.8 miles (7.7 km)]. During off-peak periods, travel time is lower on the alternate route. The annual average daily traffic (AADT) on I-35 in 1977 was approximately 79 230 in contrast to 58 140 on I-37.

The effectiveness of the CMS diversion system was studied by assessing the change in traffic volumes on the freeway, interchange ramps, and the primary off-ramps that lead to the CBD. Effectiveness of the system as perceived by police patrols was evaluated by studying the willingness of the police to use the CMSs during incidents over a period of two years.

SUMMARY OF RESULTS

Analysis of seven incident case studies for which all relevant data were available revealed that, on average, diversion rates when the CMSs were used were higher than normal but were about the same as the natural diversion that occurred due to incident congestion when the CMSs were not used.

Two factors seemed to contribute to the less-than-acceptable results:

1. The diversion ramp was too close to the final destinations of divertable (CBD-bound) drivers. Thus, the amount of time saved by taking the diversion route was probably not sufficient to encourage diversion.
2. Drivers were using routes other than the diversion route when they saw messages on the CMSs. Diversion to these other routes was not evaluated as part of the point-diversion project.

Therefore, the results do not indicate failure of the MIDS but the fact that the advice came too late under the circumstances. In addition, some drivers know better routes (from their viewpoints).

LESSONS LEARNED

The program in San Antonio was successful from several standpoints. First, it gave the San Antonio CMT, particularly members of the San Antonio Police Department (SAPD), experience with operating a CMS system. It will be invaluable in the future when more elaborate systems are designed and implemented. Second, it illustrated how interagency teamwork can accomplish corridor-management objectives. Third, it allowed the research team to observe institutional hardware and operational conditions and limitations. These observations will assist other agencies that contemplate the installation of similar systems.

Although the amount of traffic diversion attributed to the CMSs may not be overly impressive, several lessons were learned from this low-cost MIDS demonstration project that will be beneficial to others. The problems encountered with the MIDS and recommendations for future systems are discussed later in the paper.

OPERATIONAL DEVELOPMENT

Development of the CMS operations plan evolved over a period of several months and included the following activities:

1. Identification of incident characteristics,
2. Selection of sites for matrix signs,
3. Determination of existing traffic patterns,
4. Development of diversion strategies,
5. Development of candidate messages,
6. Development of operational control procedures, and
7. Training of operating personnel.

Several meetings were held between TSDHPT, SAPD, and TTI in an attempt to develop a plan that was both acceptable to the operating and enforcement agencies and incorporated available inputs of recent CMS operational guidelines.

Matrix Sign Sites

Two trailer-mounted computerized bulb-matrix CMSs (Figure 2) were used to present diversion information to northbound I-35 drivers in San Antonio. The signs, available from previous TTI research studies, provided versatility in message length, display format, and rate of display.

Messages were presented on a 4-ft 10-in (174.3-cm) high and 15-ft 4-in (467-cm) wide display board. Each of the two lines was composed of an array of 33-W incandescent light bulbs, 7 rows by 64 columns, which formed a letter height of 18 in (46 cm) with a maximum capability of 13 characters. The bulbs were protected from sun glare by a glare screen attached to the front panel of the display. Previous research by TTI (4) has shown 650 ft (198 m) to be the 85th percentile legibility distance for these signs.

The ability of displaying a message on a sign was provided to the operator through the use of a digital computer located on the front side of the trailer in an environmental cabinet (Figure 2). Messages to and from the computer were transmitted and received through a teletypewriter (TTY) (see Figure 3). The coupler is located on the side of the TTY. The sign operator dialed the number of a telephone located in the CMS computer cabinet, placed the telephone ear and mouth pieces in the coupler, and then controlled the sign with the TTY. The process would then be repeated for the second sign. Automatic dial-up cards were used to reduce the time required to operate the signs.

The human factors design guide (5) emphasized the need to install CMSs far enough upstream from decision points to allow the driver time to take appropriate action. Site selection in San Antonio was constrained somewhat by horizontal and vertical curvature and narrow right-of-way widths. The sites chosen for the two CMSs are shown in Figure 4. The first sign seen by the driver was approximately 2.2 miles (3.5 km) from the diversion point. Sign 2 was approximately 1.0 mile (1.6 km) downstream from sign 1.

Diversion Strategies

Following a review of the origin-destination (O-D) patterns, a committee that consisted of SAPD, TSDHPT, and TTI representatives mutually agreed to address only traffic bound for downtown. It was agreed that messages would be displayed only during incident conditions. Diversion messages would be displayed based on criteria of incident and end-of-queue locations (relative to the diversion point) set forth by SAPD. Messages that warn drivers of incidents would be displayed when diversion was not warranted. Estimates by SAPD, based on previous experiences, indicated that the CMSs would probably be used for diversion about once every one to two months.

A total of 120 messages (2) was initially developed by TTI. The large number of message combinations was due primarily to the desire to display detailed incident-location information.

Operational Control Procedures

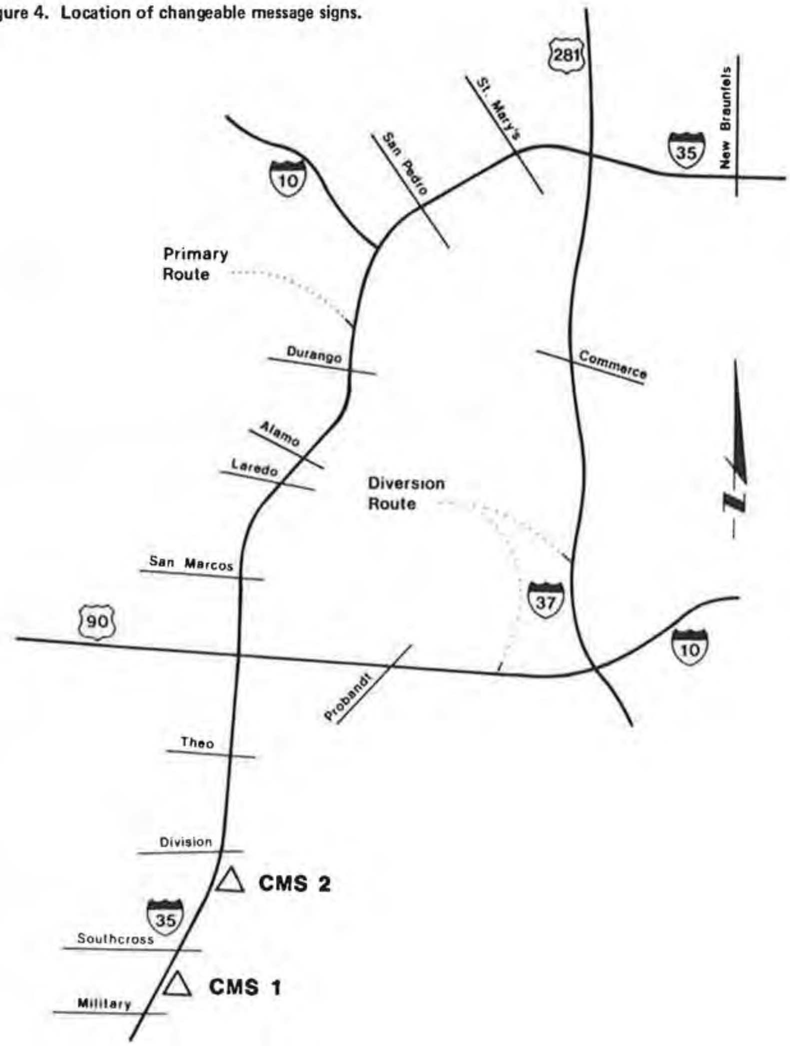
Freeway surveillance was accomplished by police freeway patrols, supplemented during peak periods with police helicopter patrols when weather permitted. Incident information and requests for sign messages were radioed to a single police dispatcher who not only dispatched police vehicles to accident scenes throughout San Antonio but also controlled the CMSs. No additional funds were available to SAPD for their participation in the CMS demonstration project. They operated the system with existing police funding and personnel constraints.

Recognizing the normally high demands placed on the dispatcher during an accident and the officers

Figure 2. Lamp matrix changeable message signs.

Figure 3. Teletypewriter with acoustical coupler.

Figure 4. Location of changeable message signs.



at an accident scene, it was necessary to develop a technique to streamline the effort involved in selecting and displaying messages. The approach developed was to assign a number to each message. Message matrices were developed for peak and off-peak periods and for complete and partial freeway blockages. The matrices allowed patrol officers and dispatchers to select numbers for the appropriate messages based on the locations of the accident and the end of queue.

A step-by-step dispatcher's procedure for operating the CMSs was also developed. The procedure listed 26 steps that were required to display a message on both signs and 26 steps to turn the signs off.

The planned scenario of operations was as follows. When a freeway patrol officer noticed unusual congestion, he or she would drive to the scene of the incident. (The helicopter pilot would fly to view the scene.) The patrol officer would either have some idea as to the location of the back of the queue or would obtain this information from another patrol officer or the helicopter pilot. The patrol officer would then look at the appropriate matrix and request that the dispatcher display the message that coincides with the message number.

By using the message chart the dispatcher would then identify the specific computer message number for each sign. He or she would dial sign 1 and display the appropriate message, and then dial sign 2 and display the message.

It is important to note that due to the software design and storage limitations of the CMS computers and the desire by TTI to use 120 messages, the message numbers and the computer storage numbers for the messages were different. For example, if an accident occurred at Alamo that blocked one lane during the morning peak period and the queue extended to I-10E, then message 32 would be displayed. The dispatcher would then look in the dispatcher's guide to find that in order to display message 32, a command must be sent to sign 1 to display message D-15 and to sign 2 to display message D-6. As the queue increased or dissipated, other patrol officers on their way to assist the officer now at the scene would notify the dispatcher of a new message number if required.

TRAFFIC DIVERSION

The primary objective of this phase of the research was to evaluate the effectiveness of the CMS system in diverting traffic to the diversion freeway route during incident conditions. A secondary objective was to develop a practical evaluation approach that can be implemented by city and state highway agencies in evaluating similar CMS systems, considering normal personnel and funding restraints.

Approach and Initial Observations

Collection of evaluation data posed several particularly difficult problems. License plate O-D surveys have been found to be a most accurate method of determining effectiveness of real-time displays. This type of study is particularly well-suited to predictable occurrences such as maintenance activities and special events (6,7). However, the random nature of incidents precludes keeping a license plate data-collection crew on standby. Therefore, a network of traffic-volume counters was employed to obtain data that could be used in the evaluation.

Analysis of data from previous TTI studies in Dallas (7) had indicated that for point diversion during special events, changes in ramp volume at the diversion point were directly related to total diversion. It was hoped that similar results could be obtained in San Antonio from strategically locating counters on primary and diversion routes. It was initially envisioned that volumes on the diversion freeway route would be expected to increase during intervals that the signs were on while volumes on the primary route would decrease.

Traffic counters were installed on I-35 at one freeway location upstream from the diversion point, on three interchange ramps along the diversion route, and on the Durango Boulevard and Commerce Street exit ramps that lead to the CBD to supplement the existing four permanent TSDHPT counters located on the primary and diversion freeway routes. The six new counters were modified to record volumes on punched tape at 5-min intervals. The TSDHPT permanent counters provided hourly counts.

Initial plans were to evaluate the volume data collected from all six of the new automatic counters. It was expected that significantly high diversion rates would be reflected by lower volumes on the Durango ramp with corresponding increases on the diversion route interchange ramps (I-35/I-10E, I-10E/I-37, and I-37/I-35) and the Commerce ramp. Statistical analyses were to be performed on the data from each ramp to test whether there were differences during each incident period compared with periods immediately preceding and following it. Evaluations of data collected during selected incident cases coupled with a thorough assessment of available data resulted in changes to these plans. In addition to the counter problems, volume changes on the Durango, I-10E/I-37, I-37/I-35, and Commerce ramps were small in comparison to total ramp volumes. Thus, it was difficult to determine whether the changes were due to the CMSs or to random variations in traffic demands. Also, the counting scheme employed did not provide a closed system whereby all input and output points were counted. With small changes in volumes on the ramps under study, it was difficult to trace origins with any certainty. For example, volume increases on the I-10E/I-37 ramp could originate from northbound I-35, eastbound US-90, and eastbound I-10. The amount of traffic that originated from I-35 could not be determined.

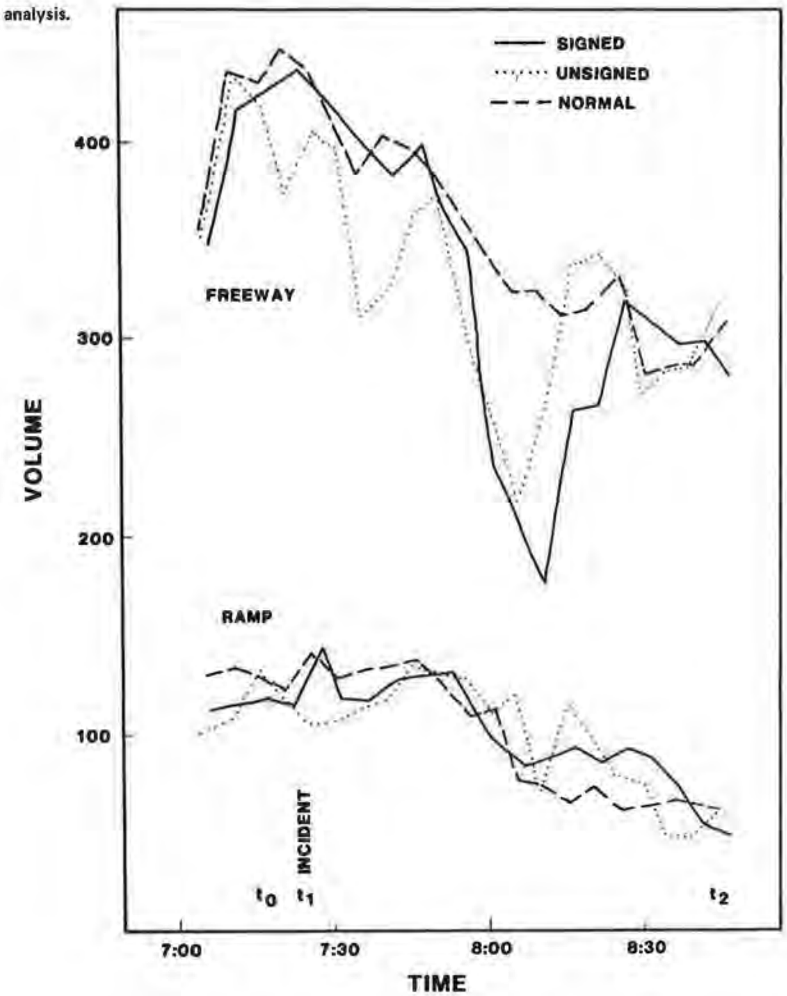
It was reasoned, however, that if there indeed was diversion due to the CMSs, the volumes on the I-35/I-10E diversion ramp would be the most sensitive to any changes and, coupled with the freeway counts made at I-35 at Theo Avenue, would at least provide some trends that indicated the effectiveness of the CMSs. Thus, efforts were then concentrated in analyzing the data from the diversion ramp and the freeway.

In contrast to the Dallas diversion studies (7) where congestion did not occur between the CMSs and the diversion ramp, queue buildup upstream from the diversion ramp due to incident bottlenecks and peak-period demand-capacity characteristics had to be considered.

The queue buildup had to be considered to accurately measure the diversion rate on the I-35/I-10E diversion ramp. It was important that motorists who read the CMSs and took the diversion route were accounted for even though they were trapped in the backup and their arrival to the diversion ramp was delayed. Therefore, the analysis period for each incident must begin prior to the time when there was a significant reduction in volumes upstream from the diversion ramp compared with normal days that indicated traffic backup from the incident. The analysis period extends to the time when congestion clears and the freeway volumes on the incident day return to normal.

Experience (6,8,9) has shown that there is a significant number of drivers who leave the freeway

Figure 5. Five-min volumes for diversion analysis.



(divert) upstream from their intended off-ramps whenever unusual congestion occurs, even though they do not know the cause of the problem. This type of diversion is often referred to as natural diversion and in many cases can be quite high. Therefore, the amount of natural diversion had to be considered in order to more accurately evaluate the true effects of the CMSs. The amount of natural diversion during incidents must be subtracted from the diversion that occurs when the CMSs were used to determine the added effects of the CMSs.

The available data were studied to obtain volumes for the following situations:

1. For the incident when the CMSs were used (signed incidents),
2. For an incident that occurred at approximately the same time of day and for which the CMSs were not used (unsigned incidents), and
3. Normal volumes for a comparable period that consisted of the average of two or three similar days (same day of week) within two or three weeks of the incident day.

A potential case-study incident was identified when volumes were available for all three situations. Certain criteria had to be met before the data from a signed incident day could be used in the analysis. These were as follows:

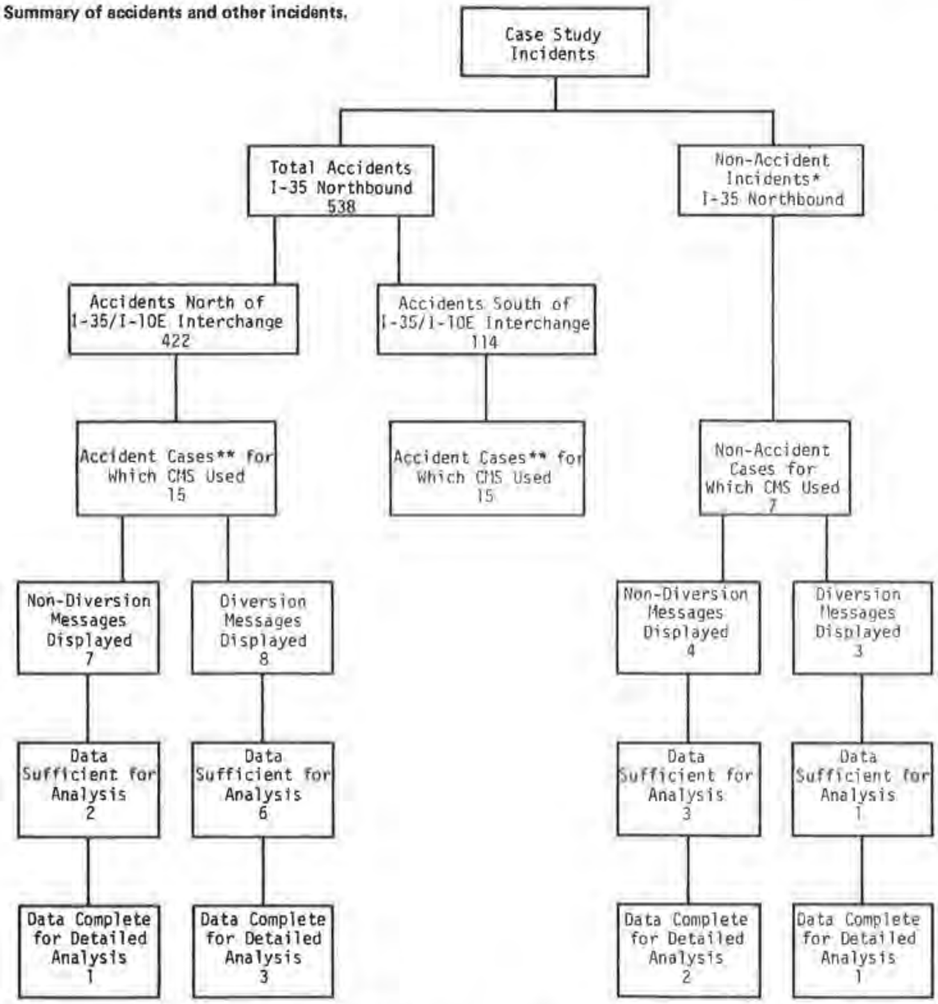
1. The incident must occur at or downstream from the diversion point,
2. The CMS must be activated, and
3. Information concerning incident time and location and the type and time of message must be available.

To develop a data base for the amount of natural diversion for each case-study incident, attempts were made to find a day when the CMSs were not used during an incident that occurred at approximately the same time of day, weekday, and within two or three weeks from the case-study incident. As would be expected, there was some difficulty in finding such data for all case-study incidents. However, as a minimum, data were found for incidents that occurred during the same year and month and reasonably close to the same time of day. In most cases, data for unsigned incidents were available on the same weekday as the case study incidents.

A normal traffic-volume data base was developed for each case-study incident by averaging data from two or three days obtained from the same time period, weekday, month, and year as the case-study incident. Care was exercised to ensure that nonincident days were selected.

The analysis process used in this project to evaluate the diversion influenced by the CMSs is illustrated in Figure 5. Freeway volumes (in 5-min increments), obtained from the automatic traffic counters located on I-35 at Theo (just upstream from the I-35/I-10E diversion ramp) and on the diversion ramp, are plotted for one of the case-study incidents.

Figure 6. Summary of accidents and other incidents.



*Spilled load, high water, stalled vehicle, etc. Total number of such incidents are not available.

**In some cases two accidents occurred.

An examination of the freeway volumes in Figure 5 shows that at time t_0 the volumes are approximately the same for all three situations. After the incident at time t_1, traffic was stored due to the demands that exceeded the incident bottleneck capacity. As the queue propagated upstream and passed through the I-35/I-10E interchange and then over the freeway detectors at Theo, the volumes significantly decreased. The drop in volumes represents the queue buildup on the freeway. When this occurred, drivers destined for the I-35/I-10E diversion ramp (both those who normally use the ramp and those who intended to use it because of the congestion and CMS messages) were delayed in both time and space. When the incident vehicles were removed from the freeway the capacity increased and the buildup dissipated. This is illustrated by the volume increase at the freeway counter station. Once the queue dissipated from the interchange area, the volumes returned to those normally expected for the particular time of day. The volumes for all three situations are approximately equal at time t_2. In order to account for traffic volume by using the I-35/I-10 diversion ramp for all three situations, the analysis included the time period from t_0 to t_2.

Because traffic demands are likely to be different for each of three situations due to normal traffic variations, a direct comparison of volume changes on the diversion ramp for each of three situations is inappropriate. To account for normal traffic variations, volumes on the I-35/I-10E ramp were converted to percentages of the I-35 freeway demand volumes for further analysis. A basic assumption underlying this approach is that the percentage of I-35 drivers who would normally use the I-35/I-10E ramp between times t_0 and t_2 is the same each nonincident day. Increases in traffic percentages on the diversion ramp would be attributed to the incident and the CMSs.

Results

Figure 6 is a summary of the accidents and other incidents, CMS use, and resulting case-study incidents that were available for study. Details of CMS use are discussed in a later section.

As shown in Figure 6, 538 accidents occurred on northbound I-35 in the study area during the two-year study period; 422 accidents occurred at or downstream of the diversion point. The CMSs were used during 15 of these accidents. Diversion messages were displayed eight times and warning messages during seven incidents. Further analysis revealed that the necessary data for more detailed

Table 1. Results of case-study incidents.

Incident				Message Type	Analysis Period	Northbound I-35 Drivers Using Diversion Ramp		Test of Significance[a]
No.	Date	Time	Location			Condition	Percent	
1	8/17/78	8:45 a.m.	I-35 at I-35/I-10E interchange	Diversion	8:50-9:50 a.m.	Normal	19	
						Unsigned incident	22	a
						Signed incident	32	a,b
2	10/10/78	6:45 a.m.	I-10W at Colorado	Diversion	7:30-9:30 a.m.	Normal	24	
		8:00 a.m.	I-35 at I-35/I-10 interchange	Diversion	7:30-9:30 a.m.	Unsigned incident	27	a
						Signed incident	29	a,b
3	9/29/78	5:25 p.m.	I-35 at Commerce	Diversion	5:25-7:00 p.m.	Normal	25	
						Unsigned incident	24	
						Signed incident	25	
4	10/21/78	2:25 a.m.	I-35 at Alamo	Diversion	2:20-3:30 p.m.	Normal	22	
						Unsigned incident	22	
						Signed incident	22	
5	10/2/78	8:00 a.m.	I-35 at Alamo	Warning	7:50-8:50 a.m.	Normal	24	
						Unsigned incident	29	a
						Signed incident	30	a
6	10/25/78	7:35 a.m.	I-35 at Durango	Warning	7:30-8:30 a.m.	Normal	27	
						Unsigned incident	32	a,c
						Signed incident	29	a
7	11/2/78	7:35 a.m.	I-35 at Stockyards	Warning	7:25-8:30 a.m.	Normal	28	
						Unsigned incident	31	a,c
						Signed incident	28	
All incidents combined						Normal	25	
						Unsigned incident	27	a
						Signed incident	28	a,b
All incidents excluding no. 1						Normal	25	
						Unsigned incident	27	a
						Signed incident	27	a

[a] a = significantly greater than normal conditions, b = significantly greater than unsigned incident conditions, and c = significantly greater than signed incident conditions.

analysis were available for only four case-study incidents. Diversion messages were used during three of these cases and a warning message during the remaining one case.

The CMSs were also used during seven nonaccident incidents (e.g., spilled load, high water, stalled vehicles, etc.). The data sets were complete for three of these cases. Therefore, data sufficient for detailed analysis of diversion rates were available for only seven case-study incidents--four accident cases and three nonaccident cases.

Results of the seven case-study incidents are summarized in Table 1. Included in the table is the percentage of northbound I-35 drivers (counted at Theo) that used the diversion ramp (a) during normal conditions, (b) during an incident when the CMSs were not used (unsigned incident), and (c) during the incident when the CMSs were used (signed incident). Also shown are the results of the Z-test of proportions analyses (10), which tested differences between each of the three situations.

The results reveal that for five of the seven cases (numbers 1, 2, 5, 6, and 7), the percentage of traffic that used the diversion ramp during the unsigned incidents (natural diversion) was significantly higher ($p < 0.05$) than normal (on days when incidents did not occur). In four of these cases (numbers 1, 2, 5, and 6), the diversion rate during the signed incident was also significantly higher ($p < 0.05$) than normal. Of the five incident cases in which either the unsigned or signed incidents yielded greater diversion rates than what would normally be expected, only two of the cases had significantly higher diversion rates for the signed incident than the unsigned incident; in two other cases the unsigned incident yielded higher diversion rates than the signed incident. In the remaining case the diversion rates were the same for both the signed and unsigned incidents.

Combining the data for all seven incidents, the results revealed that, on average, 25 percent of the northbound I-35 traffic used the diversion ramp during the normal periods whereas 27 percent and 28 percent of the traffic used the ramp during the unsigned and signed incidents. Statistical analyses indicated that the diversion during the unsigned incidents was significantly higher ($p < 0.05$) than normal and the diversion during the signed incidents was significantly higher than both the normal periods and the unsigned incidents.

An examination of Table 1 shows that in only one of the case-study incidents did the amount of the percentage increase in diversion appear to be high numerically. During incident case 1, when the CMSs were operating, 32 percent of the traffic used the diversion ramp. This was significantly higher than both the 22 percent diversion during the unsigned incident and 19 percent during the normal period. The incident, however, was different from the others. The accident occurred on the median lane just upstream from the diversion ramp. Blockage of the lane at that point may have indirectly caused several drivers in the right lane to be trapped because of the lane drop and forced onto the diversion ramp. This may have resulted in the relatively high percentage of traffic that used the ramp in comparison to the unsigned and normal periods.

An analysis of the six incident cases excluding number 1 revealed diversion percentages of 25 percent, 27 percent, and 27 percent for the normal, unsigned, and signed incidents, respectively. The 2 percent increase in diversion during the signed and unsigned incidents was significantly higher than normal ($p < 0.05$). However, as the data show, there was no difference in the diversion rate between the signed and unsigned incidents. Therefore, on average, use of the CMSs during the incidents did not result in greater use of the diversion route than the amount of natural diversion that occurred without the signs. In addition, although the 2 percent increase is statistically significant, it is insignificant from a freeway operations standpoint.

Table 2. Case-study incidents summarized by time period.

Incident Number	Period	Northbound I-35 Drivers Using Diversion Ramp		Test of Significance[a]
		Condition	Percent	
1,2,5,6,7	Peak	Normal	25	
		Unsigned incident	28	a
		Signed incident	29	a,b
2,5,6,7[b]	Peak	Normal	26	
		Unsigned incident	29	a
		Signed incident	29	a
3,4	Off-peak	Normal	24	
		Unsigned incident	23	
		Signed incident	24	

[a]a = significantly greater than normal conditions and b = significantly greater than unsigned incident conditions.
[b]Excludes incident 1.

The data were further analyzed to determine whether diversion rates were affected by the period of day when the incidents occurred. Table 2 is a summary of the case-study incidents grouped by peak and off-peak periods.

The results reveal that diversion rates were significantly higher during the unsigned and signed peak-period incidents in comparison to what would normally be expected. However, no differences were found in the diversion rates during the off-peak periods.

A review of Table 2 also reveals that when incident case 1 is excluded from the peak-period incident analysis, the diversion during the signed incidents was higher than normal but was no different than the natural diversion that takes place during unsigned incidents.

Discussion of Results

Analysis of the effectiveness of the CMSs was based solely on a freeway-to-freeway point-diversion concept. Funds were not available and no attempts were made to evaluate traffic diversion to other arterial routes induced by the CMS messages. During conversations with a few local drivers who commute to work on northbound I-35, the drivers stated that when accident messages were displayed on the CMSs they oftentimes left the freeway via one of the off-ramps upstream from the I-35/I-10E interchange (diversion ramp) and took another route to work. These alternate routes were more convenient than the diversion route recommended by the CMSs. Diversion to other arterial routes was also frequently noticed by freeway patrol officers. Thus, there was more diversion within the system than the data indicated. These observations suggest that, in contrast to diversion of special-event traffic when drivers are willing to use the recommended alternate route (7,11), point diversion during accidents in an urban area may be a misnomer.

Special-event traffic generally is destined to the same place and, in many cases, to the same parking facility. In contrast, the destinations of commuters going to the CBD are scattered throughout the downtown area. Thus, the route recommended by the CMSs may be the most logical one but may not necessarily be the most convenient one for many drivers. They may elect to choose their own routes based on the time of day, location of the accident, the degree of freeway congestion, etc., or they may decide to remain on the freeway for one reason or another, which includes simply a reluctance to use another route.

Driver response to real-time information in San Antonio appeared to be similar to that experienced in Dallas (6), Los Angeles (8), and Minneapolis (12). Rather than diverting via one major ramp when they were notified of an incident or when they encountered unusual congestion, drivers who diverted tended to use exit ramps that lead to routes most convenient to them. In retrospect, analyzing the San Antonio CMS installation as an incident-management system rather than a point-diversion system would have most likely resulted in a more complete description of traffic diversion. The limited scope of the study focused attention on the freeway-to-freeway diversion. The results strongly indicate that agencies evaluating similar systems should monitor all off-ramps that are likely to be used. Agencies should not be misled and restricted in their evaluation approach because of terminology; they should develop an evaluation approach that will assess the full impact of the CMS system if they can afford to do so.

PERCEIVED EFFECTIVENESS OF CMS SYSTEM

Background

During the first year of operation, two problems arose that had an impact on the CMS evaluation studies. First, there were numerous problems with the automatic counters, which limited the amount of useful data for estimating the amount of freeway-to-freeway diversion attributable to the CMSs. Second, the police patrol officers and the police dispatchers who operated the CMSs encountered difficulty in using the operational control procedures (i.e., selection of sign messages based on the location of accident and length of backup) developed by TTI, TSDHPT, and SAPD. As a result of discussions with SAPD, the 120 messages in the original CMS library were reduced to 7.

The potential of a small sample size due to counter malfunctions prompted the research team and the San Antonio CMT to seek alternative measures of effectiveness. It was the feeling of police administrators and some patrol officers that the CMSs were effective in improving safety in the accident area where the officers and involved motorists were located. They believed that by keeping drivers informed of freeway conditions, the CMSs help to relieve driver frustration and anxiety. As a result, drivers are less hostile when passing an officer directing traffic. It was their opinion that regardless of whether or not the CMSs divert a considerable amount of traffic, these intangible effects are realized.

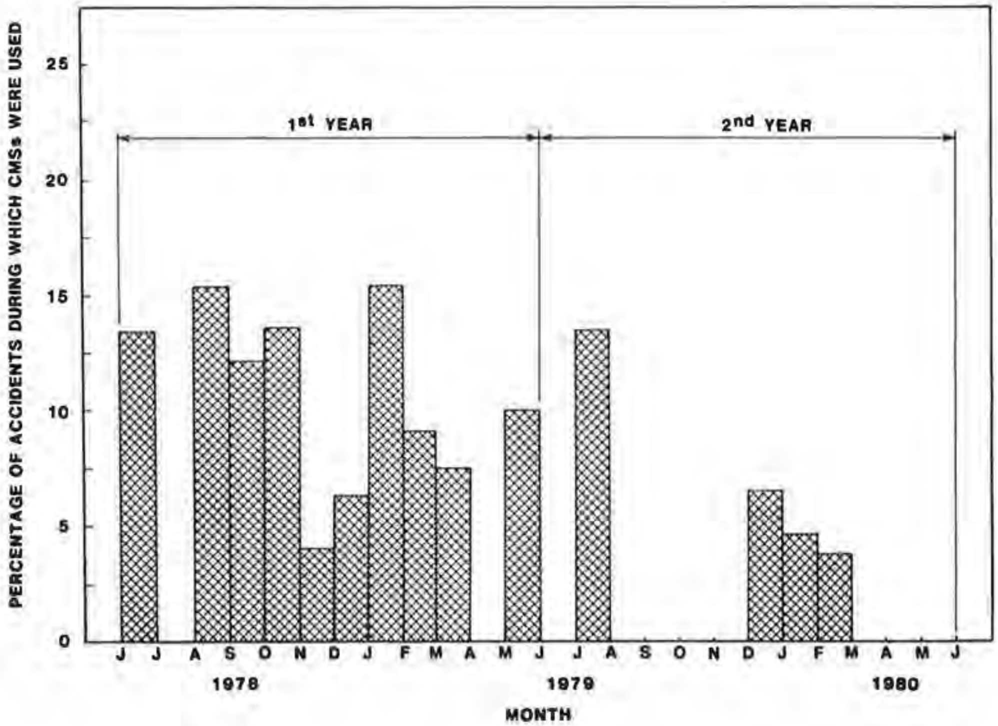
Administrators from SAPD stated that field patrol officers would use the CMSs if they felt the signs were helpful in controlling traffic on the freeway. Although the intangible benefits of less driver frustration and anxiety cannot be measured directly, it was speculated that perceived benefits can be assessed by measuring police use of the CMSs. Increased use during the second year would indicate positive feelings about the system by the police officers. As previously mentioned, operation of the system was simplified by TTI at the beginning of the second year of operation by reducing the number of messages and improving the operation control procedure.

The objective of this portion of the research was to measure the perceived benefits of the CMSs by evaluating CMS use during the first and second years of operation.

Approach

The approach used to assess the perceived effectiveness of the CMS system was to compare the use of the

Figure 7. Use of changeable message signs on I-35 in San Antonio.



CMSs during accidents and the attitudes of the police freeway patrols and dispatchers between the first and second year of operation. Accident records and CMS use information, discussed in detail later in the paper, were compiled during the two years. In addition, interviews were conducted with police patrol officers and dispatchers at the end of each year of operation.

Results

The table below lists the number of accidents and times the CMSs were used during each of the two study years:

Item	First Year	Second Year
No. of accidents	290	248
Frequency of sign use	26	6
Avg no. of accidents per month	24.2	20.7
Avg sign use per month	2.2	0.5
Accidents for which signs were used (%)	9.0	2.4

The data are also plotted in Figure 7 in terms of percentages by month during the two study years.

The results show that there was a significant reduction in the use of the CMSs during the second year of the study. The signs were used only 6 times (0.5 times/month) during the second year in comparison to 26 times (2.2 times/month) during the first year of operation. Considering the use per accident, the table shows that during the second year the signs were used for 2.4 percent of the accidents in comparison to 9.0 percent of the accidents the first year.

The results were disappointing considering the fact that the initial library of CMS messages was reduced from 120 to 7 to simplify operations following interviews with police patrol officers and dispatchers after the first year of operation. However, frequency of sign use during the second year (0.5 times/month) was not out of line with earlier estimates. SAPD administrators predicted prior to the CMS pilot program that, based on their experiences with the types of accidents on I-35, the signs would most likely be used an average of once every one or two months. The disappointment, therefore, stems from the indications of greater interest and desire to use the signs during the first year (even though the rate during the second year was in line with earlier SAPD predictions) and indications of reduced interest during the second year.

Interviews with SAPD administrators indicated the following reasons for reduced use:

1. Difficulties with existing hardware system,
2. Turnover and reassignment of SAPD personnel,
3. Shift rotations, and
4. Reduced direct contact between SAPD administrators and the dispatchers and patrol officers.

The CMS hardware system was primarily designed for research and is not ideally suited for operations by nontechnical personnel. This potential problem was recognized initially by TTI, TSDHPT, and SAPD, but it was hoped that some of the difficulties could be resolved.

Normal turnover and reassignments resulted in a situation where some police officers were not familiar with the objectives, design, and operation of the CMSs. In retrospect, the research project should have been funded to periodically furnish training to the newer officers.

The dispatchers and CMS operators were on rotating shifts. Use of the CMSs was highest during the morning peak periods. The dispatchers who became somewhat familiar with operating the CMSs would switch shifts and not return to the morning shift until two months later. The complexity of the hardware resulted in the operators being more reluctant to use the signs when they returned. CMS operating procedures were forgotten during the long periods away from operating the signs. Dispatchers switching from night to morning shifts seemed to forget the operating procedures because of the extended period between the training school and actual hands-on operations.

Probably the factor that had the greatest impact on the reduced use of the CMSs during the second year resulted indirectly from the energy situation. As was the case with other agencies, the City of San Antonio was hit by higher fuel costs. As a conservation measure, the city manager issued a directive in early June 1979 stating that official vehicles were no longer allowed to be driven to and from home. Radio communication between police administrators and supervisors with dispatchers and freeway patrol officers during the first year who requested the use of the CMSs was a positive indication and assurance of the importance of the system. The absence of radio communications between administrators, supervisors, and patrol officers was an influencing factor in the reduced use of the CMSs during the second year, according to SAPD officials.

CMS HARDWARE AND OPERATION PROBLEMS AND RECOMMENDATIONS

Operator's Control Console

The remote-control console used in San Antonio was a TTY. Although TTI researchers had no problems with the TTY while operating three trailer-mounted CMSs in Dallas (6), some police dispatchers seemed to be apprehensive about the equipment. The problem was compounded by the need to punch a "D" on the keyboard followed by a number. Some of the dispatchers lacked the confidence that the number they punched would display the desired message, even though they had a message number chart available.

There were also many occasions of dispatcher apprehension about whether the message requested was actually displayed. Although the message "D-number" was printed by the TTY printer when a message was displayed, the message content was not. This added to operational uncertainties.

The amount of operator action to display a sign message after a sign was contacted was excessive. As many as seven buttons on the keyboard had to be depressed merely to display one message and to have the D-number and computer clock time printed.

The number of CMSs that can be efficiently controlled with a TTY is also important to consider. TTI personnel had no problems operating three CMSs in the Dallas system. However, it is doubtful whether one technically trained individual could effectively and efficiently operate more than three signs in an urban area by using a TTY as a control console, even though he or she could devote full attention to sign control. Traffic conditions change too rapidly.

Thus, the following points are recommended. A TTY remote-control console can probably be effectively used in an urban area CMS system to control up to three CMSs by a technically oriented individual, provided he or she can devote full attention to operating the signs that need to be activated. A push-button console should be used when the system is operated by local or state police or nontechnical personnel, or the system has more than three CMSs. The push buttons should contain the specific CMS message that will be displayed when the buttons are depressed.

Positive visual-message verification should be provided. A message-display board should be available when either the signs are operated by nontechnical personnel or when there is a large number of signs to control. The message-display board must allow the operator to quickly identify the exact message content and the freeway locations where messages are displayed. Simultaneous display of the information is desirable. Technically oriented operators could get by with a cathode-ray tube (CRT) display, provided the number of signs is small.

Operator Considerations

Reports (5,8) have cited factors such as operator boredom as critical in the effective operation of CMS systems. Although incidents are random and there may be long intervals when the signs are not needed, the preparedness and alertness of the operator must not significantly diminish. Operator overload, rather than boredom, was a problem in San Antonio.

The CMS system operator in San Antonio timeshared responsibility with dispatching police and other emergency vehicles to locations throughout the city. Needless to say, during peak periods when the need for the CMSs was the greatest, the dispatcher was very busy responding to incidents. Overload in these critical situations required that the operator prioritize his or her tasks. Operation of the CMSs was of lower priority.

One major problem that arose in San Antonio was that because of the infrequent use of the signs by specific dispatchers due to shift rotations (signs are most frequently needed during peak periods) and other factors, the operators' self-confidence in the ability to operate the signs dwindled over time. This, in part, was a contributing factor to the decline in the use of the signs during the second year of operation. No provisions were made in the research project to retrain the operators.

Thus, the following points are recommended. The operator should be able to devote full attention to CMS operation during the peak-traffic periods when incidents are most likely to occur. During off-peak periods other related tasks are advisable, but the operator must be in a position to devote full attention to the signs when an incident occurs.

The operator should have a strong working knowledge of the freeway and streets in the corridor influenced by the CMSs. This knowledge will permit him or her to more efficiently and effectively select the appropriate information options for display.

The operator must be well-trained and confident about his or her ability to operate the system. Recognizing that sign use in smaller metropolitan or rural areas may be infrequent, provisions should be made to retrain the operators and to practice sign operation under simulated conditions. The CMS control console and associated hardware and software should be designed to allow operators to go through the actual motions of operating the system and seeing the messages appear on the confirmation panel without the messages actually being displayed on the signs in the field. These simulations should be conducted with a supervisor at least every six months. The operators should be encouraged to practice the simulated operations on their own at more frequent intervals.

System Operation

The decision was made by local highway and police agencies that the San Antonio CMS system would be operated by SAPD. SAPD administrators and supervisors were enthusiastic supporters and lent considerable encouragement for this arrangement. Many institutional, personnel, and funding constraints limited the capability of the local police to staff the system to the levels needed to maintain an effective system during the two-year study. However, SAPD believes that the police should have operational responsibility.

Thus, the following point is recommended. Some local police departments are in a position to assume responsibility for operating MIDSs in urban areas. When the system is to be operated by local or state highway agencies, the police should be involved with the planning and design of the system and must be an involved partner when the system is operated.

Telecommunications

As previously discussed, the operator's console (TTY) in San Antonio communicated with each sign by way of a telephone dial-up system. This required that a sign be called before a message could be displayed, changed, or removed. Although the amount of time required would not be excessive and the efficiency of operation would not be seriously affected for a two-sign system with an experienced operator, problems could arise with larger urban area systems or when operators are not proficient in the use of the CMS system. Experiences with the telephone dial-up system in San Antonio indicated that it was quite inadequate for the occasional user who had a multitude of other simultaneous responsibilities.

Thus, the following point is recommended. It appears that the telephone dial-up system may be adequate for a small number of isolated CMSs in urban areas or for small CMS systems in rural areas. However, most urban systems should employ other telecommunications techniques to minimize the time required to change messages on the signs.

Surveillance

Surveillance is required for incident detection and an assessment of the operating conditions in the corridor. Detection of incidents, especially during peak periods, posed very little problem in this study. The thoroughness with which SAPD covered the freeway system with ground and air units reduced incident detection time to the lowest time possible without extensive freeway instrumentation. For maximum effectiveness, incidents should be detected rapidly enough to allow the initiation of diversion before the exits to alternate routes are blocked by the queue from the incident.

Accurate identification of the incident location is also critical. Selection of messages is highly dependent on the location of the incident. The more specific the description of the incident in the CMS messages, the more critical the identification of the incident location becomes. For example, ACCIDENT AT DURANGO requires a much more accurate location determination than does ACCIDENT NORTH OF US-90.

A second important function of surveillance is to provide information about conditions in the corridor. In San Antonio, the dispatcher-sign operator had to rely on those officers in the field to describe the conditions on the freeway. The patrol officers were in most cases so busy with investigating the incident and moving traffic that they were not able to provide this information to the operator. Thus, the operator was required to blindly operate the CMS without having the assurance and confidence that the messages displayed were the correct ones for the existing conditions. Eventually, some of the operators decided not to use the signs.

Thus, the following points are recommended. Use of police patrols is a good way to identify the occurrence and location of freeway incidents in small metropolitan areas. It is not adequate to provide detailed information that concerns the traffic conditions on the freeway so that the operator can make appropriate decisions about the messages. Electronic detector surveillance complemented by closed-circuit television are necessary parts of a CMS system in urban areas.

Discussion

Glen C. Carlson

Most, if not all, administrators of freeway traffic-management programs feel that motorist information services are an important system element. A design team that develops a MIDS must make a wide range of decisions on the type of hardware to be used, location of the field installations, surveillance techniques, telecommunications equipment, control equipment, and operational policies and strategies. Primary considerations in making these decisions include the following:

1. System effectiveness--Effectiveness in terms of reducing the number of secondary collisions that follow incidents, encouraging motorists to use alternate routes when appropriate, and reducing driver tension and anxiety. Accomplishing these objectives would obviously reduce fuel consumption and air pollutant emissions.
2. Reliability--The ability of the hardware to function properly without placing an inordinate burden on staff and financial resources.
3. Operations--Policies and strategies must be developed to permit efficient system operation without creating excessive demand on the operator's time.
4. Cost--Cost items include capital, operating, and maintenance costs relative to benefits derived.

Dudek, Stockton, and Hatcher did not provide information on system costs or maintenance experience for the San Antonio system, presumably because the CMS hardware consisted of trailer-mounted units available from previous studies. However, the study results relative to system effectiveness and operations experience were very interesting and should prove to be useful to other agencies. The primary evaluation item was the diversion of freeway traffic to an alternate freeway route, and a secondary evaluation item was the amount of MIDS use by the operating agency. They also presented recommendations on CMS hardware, system operations techniques, and evaluation efforts for future systems.

DIVERSION STUDIES

To evaluate the effectiveness of the MIDS in diverting traffic to the alternate freeway route, data from seven case-study incidents were studied. Analysis of the results indicated that use of the CMSs during incidents did not result in greater use of the diversion route than the amount of natural diversion that occurred without the signs. The authors conclude that this does not indicate failure of the MIDS but that the message came too late and some drivers apparently knew better alternate routes. This conclusion seems to be valid because informal discussions with local drivers and feedback from freeway patrol officers indicated that diversion to arterial street alternate routes may have been considerable.

The problems that the researchers had with the diversion studies illustrate the difficulties that can be encountered in evaluating diversion systems. A recommendation is made that agencies evaluating similar systems should monitor all exit ramps that are likely to be used. This is a good recommendation because a systemwide evaluation is desirable; however, since case studies are usually used to evaluate diversion systems, there are several potential problems that may arise, which include the following:

1. Incidents occur at unpredictable intervals and it may be necessary to extend the study over a lengthy time period to obtain an adequate sample.
2. To provide systemwide data collection over an extended time period, it is desirable to have the freeway and ramps heavily instrumented with permanent detectors. This type of instrumentation is often not available.
3. Even if permanent detectors are available, they will be subject to failure, and the exit-ramp detectors will probably have a lower maintenance priority than other detectors more critical to system operation.
4. If portable counting equipment is used it will have to be left in the field for lengthy periods of time and will require a great deal of staff time for collecting data, making adjustments, and recharging batteries.
5. Regardless of the type of detection used, it may be difficult to predict how far upstream the diversions start. For example, motorists learning of an incident via commercial radio stations or citizen band (CB) radio may stay off the freeway altogether.

PERCEIVED EFFECTIVENESS

The second evaluation item described was the effectiveness of the system as perceived by police freeway patrols and dispatchers. This was studied by collecting data on the willingness of the police to use the CMSs by comparing the first and second years of operation. Study results showed that there was a significant reduction in the use of the CMSs during the second year. The authors concluded that the reduced use was not caused by a perceived ineffectiveness of the system but rather was attributable primarily to a loss of radio communication between SAPD administrators and dispatchers.

A review of the data in Figure 7, which uses a slightly different perspective, seems to support the authors' conclusion. If system operators perceived that the CMSs were not effective, it probably would not take them an entire year to reach that perception. Yet the data in Figure 7 show that the CMSs were used fairly consistently over the first 14 months of operation. During the first seven months, for example, they were used during 9.1 percent of the accidents while during the next seven-month period they were used during 7.9 percent of the accidents. Over the final 10 months of the two-year study period, however, the CMSs were used during only 1.4 percent of the accidents. This seems to support the authors' conclusion that factors other than the perceived effectiveness of the system led to the reduced use.

CMS HARDWARE AND OPERATIONS RECOMMENDATIONS

The recommendations regarding the operator's control console, operator considerations, and system operation are well-grounded based on the study findings in San Antonio. Many of these findings have been affirmed by the experiences of other agencies, several of which are referenced in the paper. There seems to be general agreement that systems may be operated successfully by either police or transportation (highway) agencies and that the key factor is to develop teamwork between agencies. The success or failure of a system can depend on this teamwork because if CMSs are not operated efficiently they will lose credibility and effectiveness.

The recommendations regarding telecommunications are also valid based on the study results and experiences of other agencies. In most cases, the type of telecommunications techniques used will probably be determined by the data-transmission requirements of other traffic-management system elements such as surveillance and ramp control.

Regarding the discussion on surveillance, the authors made a very significant recommendation on the techniques used for incident detection and CMS operation. They reach the conclusion that "electronic detector surveillance complemented by closed-circuit television are necessary parts of a CMS system in urban areas." This type of surveillance is indeed necessary if CMSs are to be operated at maximum efficiency in large urban areas. Exceptions where this sophisticated electronic surveillance may not be needed include agencies that use monitoring via helicopter or a fleet of emergency service vehicles.

SUMMARY

Overall, this paper did an excellent job of describing the San Antonio MIDS evaluation, providing a frank discussion of the problems encountered and the adjustments made to compensate for them. The authors have made recommendations on system design, operations, and evaluation that should prove very useful to other agencies.

A final thought that warrants brief discussion regarding this research effort is the establishment of quantifiable goals for CMS systems. The authors did not approach the evaluation task with a set of quantifiable goals to be used in determining system effectiveness. There is often no valid basis to establish such goals for a CMS system, and this appears to have been the case with the San Antonio project. Setting arbitrary goals can lead to false expectations for a project and can also lead to misinterpretation of the study results if they are not met.

E.R. Case

Despite the fact that Dudek, Stockton, and Hatcher have concluded that the results of the diversion project described in their paper are less than acceptable, many important lessons were learned that will be invaluable to those involved in the planning and design of similar systems in the future. Probably the most important of these is that it is essential to use a systems approach in developing such systems. Incident management certainly does imply a much wider scope than simple single-point diversion, and this should be reflected in both system design and evaluation.

The paper has raised a number of important issues that are of critical interest to those contemplating the implementation of traffic-management systems that depend on the predictability of driver response to traffic-information systems such as CMSs. Although many of these have been addressed since this project was conceived in 1977, there are still a number of areas where more research and development are required. One is a human factors issue that relates to determining what could be called the motorists "marginal propensity to divert" under various conditions. Another relates to the use of traffic simulations to predict system performance. A variety of simulation models are available (13), which are increasingly being used for system design (and even real-time control). Data collection remains a problem, however.

In my view there are some important lessons to be learned from this project with respect to natural diversion. Natural diversion is a driver behavior pattern that has developed over a period of time in response to a variety of incident conditions. It can only really be measured accurately by an extensive survey that uses questionnaires and yet it is essential information for the development of a freeway corridor traffic-management system. Natural diversion patterns may well have to be modified to implement an optimal freeway corridor diversion strategy.

In the present case, the results indicate a relatively high degree of natural diversion that appears to be very responsive to the onset of congestion. It seems that a significant number of CBD-bound drivers were already receiving sufficient and timely traffic information--from whatever source--to enable them to make a decision to divert. This would certainly tend to reduce the effectiveness of the CMS messages, as was observed. No mention was made in the paper of commercial-radio traffic reports, but perhaps they played a role here.

The results obtained clearly support the authors' contention that single-point diversion is a myth in this particular case and indeed is probably generally so except in certain highly contained situations. But this does not necessarily mean that systems performance can only be evaluated by monitoring all possible diversion ramps. If the goal of diversion (or incident management) is to alleviate congestion and reduce accidents, then these could be used directly as system measures of effectiveness, regardless of where the diversion occurs. For example, the maximum length of the mainline queue due to an incident and the time it takes to dissipate are certainly valid measures of the degree of freeway congestion and are not too difficult to obtain. Also, the reduction in secondary accidents is a direct measure of overall system effectiveness.

Only 7 of the original 120 messages were used in the second year to simplify operations. No mention is made of which ones were chosen or the basis on which the choice was made. I am wondering, in view of the authors' extensive background in this area, if there were any human factors considerations involved that related to the perceived effectiveness of the messages?

Many of the problems experienced by the authors would not be experienced today where the trend is to implement CMS systems as an integrated part of a freeway corridor traffic-management system that may employ other strategies in addition to diversion. Such systems include extensive electronic detector surveillance as well as closed-circuit television (CCTV). Many employ automatic CMS message selection based on traffic information supplied by the electronic surveillance system to implement optimal incident-management diversion strategies. Nevertheless, there will still be frequent occasions where CMSs must be controlled manually, and the guidelines provided in the paper will be useful in defining acceptable operator work load levels.

In 1979, the Ontario Ministry of Transportation and Communications installed a CMS on the Queen Elizabeth Way (QEW) west of Toronto (14), with the dual purpose of evaluating its effectiveness in controlling traffic and obtaining experience in the installation, operations, and maintenance of this type of sign. The CMS system is integrated into the QEW Freeway Surveillance and Control System (FSCS) (15) and is manually controlled from the traffic control center about 3.7 miles (6 km) to the east.

Although the CMS is installed just upstream of a major interchange, the performance evaluation was limited to mainline control because the local political situation precluded any attempt to divert freeway traffic onto neighboring arteries (mainly residential). The results are therefore not entirely relevant to the subject of this paper, but they still may be of interest to those contemplating the use of CMSs for traffic control.

The QEW CMS is a magnetic disc matrix type with 2 lines of 22 characters each. It extends across all three lanes and replaces an existing static sign just upstream of a major interchange. Each of the character modules is 18 in (46 cm) high and is comprised of a 5x7 matrix of 2-in (5-cm) diameter fluorescent yellow discs. It is front lighted for visibility at night.

About 35 preprogrammed messages are available, although only about 7 are used routinely on a daily basis. Other messages can be typed in for special situations. The sign is available for use 24-h/day and is controlled through the QEW FSCS during the morning peak and by the Ontario Provincial Police at other times. CCTV is available to monitor the CMS messages and the resulting driver behavior.

The sign was evaluated under a variety of incident conditions. Typical of these is a study of the effectiveness of the CMS based on a lane-blocking incident located some 1.9 miles (3 km) downstream of the sign. A series of messages were used that advised that the lane was blocked and the resulting traffic changes were monitored by the FSCS detector station located 0.62 mile (1 km) downstream of the sign. The volume dropped more than 30 percent in the 10-min period during which the lane-closure message was shown. In other tests, it was observed that up to 50 percent of drivers followed the advisory messages on the sign.

Next year the Ministry plans to install a similar CMS at a major freeway-to-freeway diversion point on the QEW. The lessons learned in this paper will be of great value in the planning, design, and evaluation of this system.

Authors' Closure

We are appreciative of the thorough review and critique by Carlson and Case. Their comments, based on vast hands-on personal experiences, add significantly not only to the paper but also to freeway incident-management technology.

The establishment of goals for a CMS or other type of incident-management system is important. However, operating agencies are handicapped because of the difficulty in (a) predicting effects and (b) measuring effects.

It is difficult for operating agencies to quantify CMS system goals in terms of the percentage of anticipated reductions in accidents (both primary and secondary) and congestion and increases in diversion because there is a lack of available published data that can be used as a basis for estimating potential effects. Published field evaluations of CMS systems are very sparse and a data base sufficient for the agencies to use for their estimates is not available.

Two goals that most likely would be embraced by agencies are to alleviate congestion and reduce secondary accidents. Unfortunately, these are difficult to measure in a real-world setting for CMSs used for accidents. Congestion is often determined by measuring the length and duration of traffic queues that result from the incidents. Although the data would not be difficult to obtain for these to be valid measurements, all incidents must have the same characteristics in terms of location, type, degree, and duration. The wide variety in incident characteristics prevents the operating agencies from relying on congestion measurements as a basis for determining the effects of the CMSs. This is why we decided to use the amount of diversion during the specific study period as discussed in the paper. In addition, the number of known secondary accidents on a section of highway is normally too small to obtain statistically significant results.

The original set of 120 messages was reduced to 7 in the second year to simplify operations. Ideally, drivers should be told where the incident is. In San Antonio, incident location was initially referred to existing major street crossings, which accounted for most of the messages. After the first year, the I-35/I-10E interchange was used for referencing the incidents. Although the accident locations were not specific, we believe that the degree of response to the messages in terms of diversion was not adversely affected. The word ACCIDENT on a CMS has a profound effect on the driver's decision to divert. It is even more profound when MAJOR ACCIDENT is displayed.

REFERENCES

1. W.R. Stockton, C.L. Dudek, and D.R. Hatcher. Evaluation of the I-35 Route Redesignation in San Antonio. TRB, Transportation Research Record 722, 1979, pp. 77-83.
2. C.L. Dudek, W.R. Stockton, and D.R. Hatcher. San Antonio Motorist Information and Diversion System. FHWA, Rept. FHWA-RD-81/018, Dec. 1980.
3. Major Point Diversion Demonstration Program, Austin-Laredo Corridor--I-35 in San Antonio, Route Redesignation. Texas State Department of Highways and Public Transportation, Austin, July 1976.
4. W.R. Stockton and C.L. Dudek. Legibility Study of a Lamp Matrix Sign. TRB, Transportation Research Record 683, 1978, pp. 4-5.
5. C.L. Dudek and others. Human Factors Requirements for Real-Time Motorist Information Displays: Volume 1, Design Guide. FHWA, Rept. FHWA-RD-78-5, Sept. 1978.
6. J.D. Carvell, J.M. Turner, and C.L. Dudek. Human Factors Requirements for Real-Time Motorist Information Displays: Volume 15, Freeway Incident Management Studies. FHWA, Rept. FHWA-RD-78-19, Aug. 1978.
7. G.D. Weaver, S.H. Richards, D.R. Hatcher, and C.L. Dudek. Human Factors Requirements for Real-Time Information Displays: Volume 14, Point Diversion for Special Events--Field Studies. FHWA, Rept. FHWA-RD-78-18, Aug. 1978.
8. M.A. Boganoff and R.P. Thompson. Evaluation of Warning and Information Systems: Part I, Changeable Message Signs. California Department of Transportation, Sacramento, Freeway Operations Branch Rept. 75-5, July 1976.
9. W.R. Stockton, C.L. Dudek, and D.B. Fambro. Evaluation of a Changeable Message Sign System on the Inbound Gulf Freeway. Texas Transportation Institute, Texas A&M Univ., College Station, Rept. 200-1F, Aug. 1975.
10. W.J. Dixon and F.J. Massey. Introduction to Statistical Analysis. McGraw-Hill, New York, 1969.
11. J.C. Dabney and C.L. Dudek. Driver Response to a Highway Advisory Radio System in New Braunfels, Texas. TRB, Transportation Research Record 808, 1981, pp. 38-40.
12. G.C. Carlson, R.D. Dahl, and A.Z. Lari. Evaluation of Highway Advisory Radio in the I-35 Traffic Management Network. FHWA, Rept. FHWA-RD-79-33, March 1979.
13. B.L. Allen, S.M. Easa, and E.R. Case. Application of Freeway-Corridor Assignment and Control Model. TRB, Transportation Research Record 682, 1978, pp. 76-84.
14. M.J. Young and K.M. Williams. Changeable Message Sign--QEW, Mississauga. Presented at the Annual Meeting of the Roads and Transportation Association of Canada, Toronto, Sept. 1980.
14. E.R. Case and K.M. Williams. Queen Elizabeth Way Freeway Surveillance and Control System Demonstration Project. TRB, Transportation Research Record 682, 1978, pp. 84-93.

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Development of Compact Microsimulation for Analyzing Freeway Operations and Design

A.G.R. BULLEN

The development of the freeway microsimulation FOMIS is described and an example of the kind of analysis possible with it is given. The model uses the vehicle-behavior algorithms of the freeway component of the simulation INTRAS, which is the corridor microsimulation developed for the Federal Highway Administration. The integration of these algorithms into a revised model structure overcomes some traffic operations difficulties experienced with INTRAS, greatly improves model speed, and provides a simulation model that can run on computers of very limited capacity. As an example of its application, a weaving section on I-95 in Dade County, Florida, is analyzed. The resulting analyses indicate operating patterns not generally derivable with existing methods. Varied and unusual design solutions emerge from the analyses. A model of this kind, which uses the particular traffic algorithms of INTRAS, has a potential as a supplemental tool to established procedures for applied freeway design problems. It could also assist in research into weaving and merging behavior in complex situations.

Most freeway operations and design problems are currently analyzed with standard capacity analysis procedures (1,2) or by macroscopic computer models such as FREQ (3). In many instances, however, an additional level of analysis may be desirable if the problem involves unique or unusual geometric features and/or traffic operating characteristics. In such cases a microsimulation could provide the necessary analysis capability.

An effective freeway microsimulation with the appropriate capabilities is contained in INTRAS, the corridor-simulation model developed for the Federal Highway Administration (4). This model, however, has some drawbacks with regard to model size, running time, and availability. It would be useful, therefore, to have the unique capabilities in INTRAS available in a more compact form for application to specific freeway analysis problems.

This paper presents an example of how this might be done through the development of a compact freeway microsimulation that integrates the vehicle-behavior algorithm of INTRAS into a modified model structure. This revised model, FOMIS, can operate on a small computer system with limited capabilities and has a running time substantially faster than INTRAS. As an example of the potential use of a model of this kind, an example of an analysis of a freeway weaving section is provided.

BACKGROUND

Since most freeway analysis problems of significance involve traffic flows at or near capacity, a simulation, to be useful, must have the capability of modeling these traffic conditions. In particular, the simulation must be able to reproduce weaving and forced lane changing at high-traffic concentrations, it should be able to internally generate the breakdown from free flow to congested flow, and it should be able to replicate the wave propagations of congested flow.

The microsimulation that has these characteristics is INTRAS and, accordingly, it provides the starting point for the development of FOMIS. Unfortunately, for its applicability to many freeway analysis problems, INTRAS is primarily a research model of great size and limited availability. Thus, there appears to be a need to use the power and flexibility of its algorithms in a simpler framework more suitable for applied problem analyses.

Users of INTRAS have reported problems with some aspects of traffic behavior. These relate mainly to vehicles that merge from acceleration lanes, vehicle behavior at exit ramps, and the method of assigning destinations. Some of these relate to the complications of communication between vehicles across link boundaries. The link structure in INTRAS defines geometric conditions along the freeway. Vehicle processing in the simulation is carried out on a link-by-link, then a lane-by-lane, basis. In each time period, two complete scans of the system are needed--one to handle car following and the other to handle lane changing.

FOMIS provides a revised model structure that is intended to streamline the simulation process by restricting it to the freeway only, by eliminating the link structure, and by reducing vehicle processing to a single scan. Several of the reported vehicle-operation problems are also eliminated by the modifications. The basic vehicle-behavior algorithms, however, remain relatively unchanged and provide the central logic for this simplified model.

VEHICLE-BEHAVIOR ALGORITHMS

Vehicle behavior is modeled by car-following and lane-changing algorithms, for which the full derivations are given elsewhere (4-6). Car following is a combination of two individual algorithms. One is based on the premise that a following vehicle will always seek a desired headway, which will be a function of vehicle speed, relative speed, highway capacity, and driver and vehicle type. The other is a collision-avoidance algorithm that acts as an override on the desired following-distance equation and ensures that the vehicle is always in a safe position. The combination of these algorithms allows vehicles to be temporarily inside their desired headways; thus, the simulation can reproduce the very short headways and headway oscillations that are so prevalent in congested flow. This provides the mechanism for the internal generation of the breakdown from smooth free flow to turbulent congestion, with the associated shock-wave propagation.

The lane-changing mechanism uses the collision-avoidance algorithm, as it is assumed that the desired following behavior is not present during a lane change. The changing vehicle must satisfy the safe headway conditions for both the leader and the follower of the gap that it is moving into. The lane change is assumed to take the finite amount of time that a vehicle needs to physically change lanes, and this is accomplished in the simulation by projecting ahead to the end of the lane-change time, i.e., the final positions of the interacting vehicles. They must be in safe relative positions at that time but, during the time of the lane change itself, temporarily unsafe positions are allowed. This mechanism has been shown to replicate forced lane changing as it allows changing vehicles to crowd into otherwise nonexistent gaps in congested conditions. The lane-changing rules also allow for courteous drivers to create gaps for changing vehicles.

STRUCTURE OF FOMIS

The vehicle-behavior algorithms described above, which are the logical core of the freeway component of INTRAS, are also the basis of FOMIS. It is in the simulation structure that contains these algorithms that the major differences exist. In particular, the link-by-link description of the freeway is not used. In FOMIS the freeway is structured as a single continuous unit with all elements, whether moving vehicles or fixed objects, defined by their longitudinal distance from some fixed origin and their lateral position by lane number. Within this structure, the simulation can handle a wide range of geometrics at any desired location, which includes lane drops and adds, weaving sections, entrance and exit ramps of any number of lanes, and freeway-to-freeway merges or diverges. Structurally, FOMIS has no upper limits on the length or the number of lanes of the freeway to be modeled. The specification of array size, however, does place practical limits on these parameters.

Other characteristics that can be implemented are vehicle detectors and lane-capacity reductions caused by permanent geometric characteristics or temporary traffic incidents. These features may be placed anywhere on the freeway in any number. There is one constraint on the fixed features in that they must have a longitudinal separation such that a vehicle cannot cross two physical features in a single scanning period. In practice this means a spacing of at least 150 ft.

SIMULATION OPERATION

Vehicles are processed from downstream to upstream in the order of their physical location, regardless of lane. A single sweep is made each scanning period and all functions, including car following and lane changing, are carried out during this sweep. The moving window in which the vehicle processing is carried out contains all adjacent vehicles. This means that for lane changing, the adjacent gaps can be referenced directly whereas in INTRAS they must be searched for individually whenever they need to be referenced.

The lane-changing algorithm has been upgraded to improve high-volume weaving. In the original lane-changing mechanism, the projected behavior of the lead vehicle had to be its worst deceleration condition, which was somewhat unreasonable. Now, however, the projected behavior of the lead vehicle is based on its own leader and this gives more realistic acceleration projections during weaving.

Another change with FOMIS is its handling of vehicle destinations. In INTRAS, these are randomly assigned on a link-by-link basis. In FOMIS, an origin-destination matrix provides the distribution of destinations by lane and exit ramp for vehicles that enter each lane of each entrance ramp. This destination pattern remains consistent thoughout the length of the freeway. Once on the freeway, the lane choice of a vehicle is controlled by the destination characteristics and by two overlapping zonal influences. The first indicates whether the current lane is incompatible physically with the vehicle's destination and, if so, the distance still available to reach a compatible lane. The second set of zones gives lane desires when lane choice is not mandatory. These zones can be keyed to driver information given by signs and allow lane changing as vehicles approach their destination ramps.

Information on speeds, volumes, and densities for each detector is printed out at intervals set by the user. Individual vehicle data can also be displayed. The simulation currently runs on a DEC 10 with 20K core. This gives arrays that allow up to 8 lanes (including ramps) with a system of up to 25 lane miles of congested traffic. Under these conditions, which include heavy concentrations of weaving traffic, the model runs at about 1500 vehicles/scanning period. In comparison, INTRAS with overlays requires 500K in modeling a corridor. FOMIS runs at three to four times the speed of only the freeway component of INTRAS for the same size problem on the same DEC computer. Compared with the full INTRAS model, the running time of FOMIS is probably on the order of 10-20 times faster.

EXAMPLE APPLICATION: WEAVING SECTION DESIGN

As an example of its use, the simulation has been applied to an analysis of a freeway weaving section. For the example, analysis of a length of northbound Interstate-95 in Dade County, Florida, which runs between I-195 and I-395, was used. Figure 1 shows the schematic layout of the freeway with its peak traffic flows projected for the year 2005. The weaving section is about 5350 ft long with five lanes entering and six lanes leaving. Conventional analysis led to a weaving section of six lanes, with the sixth lane added to the right lane.

In the simulation analysis, six alternative geometric designs were tested. These, as shown by Figure 2, are as follows:

A. The existing design.

B. Adding the sixth lane on the left side so that it becomes part of the through portion of I-95.

C. Adding the sixth lane in the center so that the two merging highways are shifted a lane further apart.

D. A variation of alternative C where the start of the weaving section is delayed by a barrier of 1000 ft. Thus, the through part of I-95 is first expanded to four lanes and it then enters the weaving section, which is reduced to 4350 ft in length.

E. Similar to alternative D except that the initial expansion by one lane takes place on the I-395 approach.

F. A combination of alternatives D and E with both approaches expanding by one lane before the weave takes place. This gives a weaving section of seven lanes.

The simulation model was calibrated to a lane capacity of 2100 vehicles/h. General vehicle-type distributions and vehicle-speed distributions were used as detailed data from the site were not available. Since the analyses were comparisons between alternatives rather than the precise measurements of any one alternative, there was no real need for exact local traffic conditions. The model was validated to the degree that, when existing traffic flows were simulated on the existing geometrics, the congestion patterns known to exist were reproduced.

The alternatives were compared by simulating two conditions on each one. The first condition was the projected traffic volumes and patterns for the year 2005. The second condition was that of complete oversaturation. The total throughput of the weaving section was measured, given maximum entering volumes on all lanes and the same weaving percentages as in the first condition.

ANALYSIS RESULTS

The analyses of the year 2005 volumes indicated different congestion patterns for the various alternatives. Figure 3 shows the level of service in the weaving section for each alternative

Alternative A, which is the existing design, does not rate well in these comparisons. The primary weaving area is fully congested while another congested condition occurs in the area further downstream where I-395 traffic merges with the I-95 through traffic. The sixth lane on the right remains at level-of-service A. For most of its length, this lane is not used at all, and at the end of the weaving section it carries volumes of only 400 vehicles/h. Thus, while macro-weaving analyses indicate that a weaving section of six lanes will be sufficient, the effective weaving area is only five lanes, especially at the beginning of the weave; hence, the actual performance of this alternative is substandard.

Figure 1. Example weaving area with design volumes.

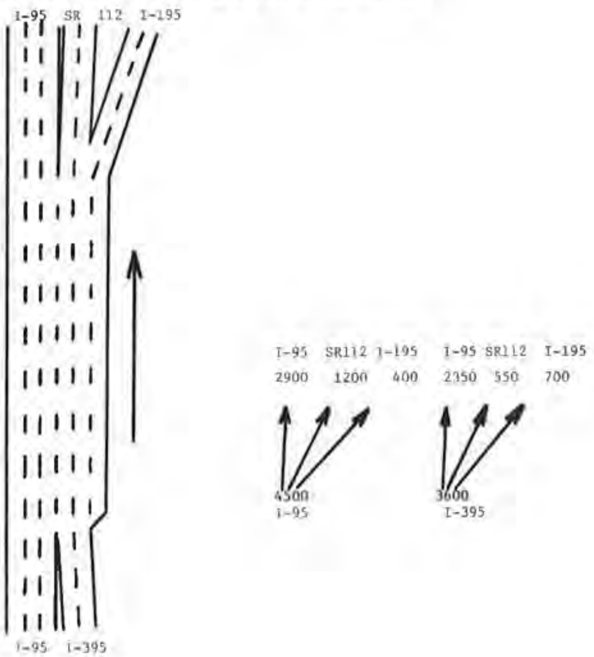
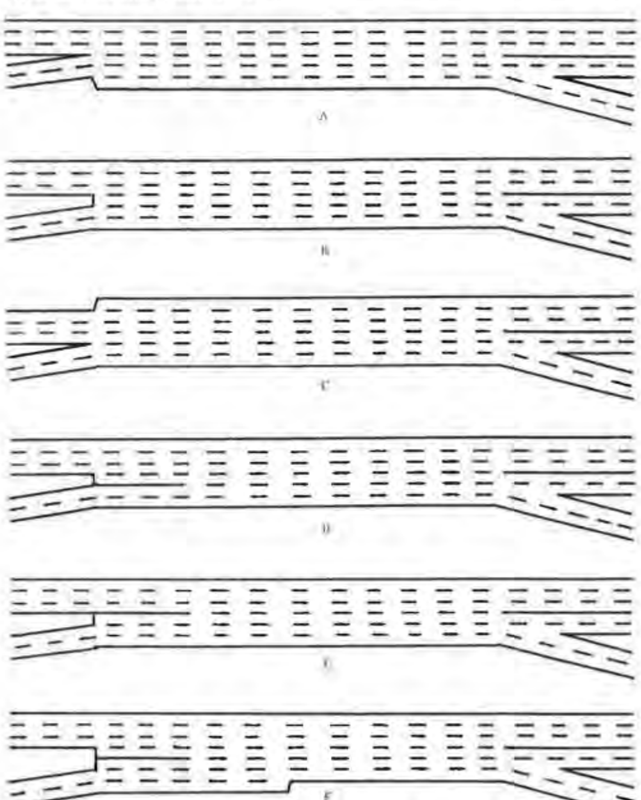


Figure 2. Six design alternatives.



Alternative C shows performance deficiencies similar to those of alternative A. Conditions on the I-395 approach to the weaving areas are worse since the weave is more difficult, while the secondary merge is somewhat better as the added lane is helping maintain the through traffic stream.

Alternative B, with the added lane in the center, is clearly a major improvement. No part of the weaving section is congested, with two limited areas at level-of-service D. In the primary weaving area, levels of service of A and B are maintained and this is the significant operational improvement over alternatives A and C, where the primary weaving area is congested for the design volumes.

Alternatives D, E, and F all show satisfactory operational levels of service, particularly in the primary weaving area. There is some congestion in the secondary merge caused by the shorter total weaving length available. The area of level-of-service D is greatest in alternative E, which constricts the main I-95 traffic flow somewhat more than the other alternatives.

With all of the alternatives, the simulation gives a detailed summary of the spatial patterns of the levels of service of traffic operations within the weaving section. The macro-analysis methods, on the other hand, give only the aggregate estimates of overall weaving section performance. This means less information on particular operational problems and less sensitivity in the analyses to detailed geometric design alternatives.

Figure 3. Operating conditions at design volumes.

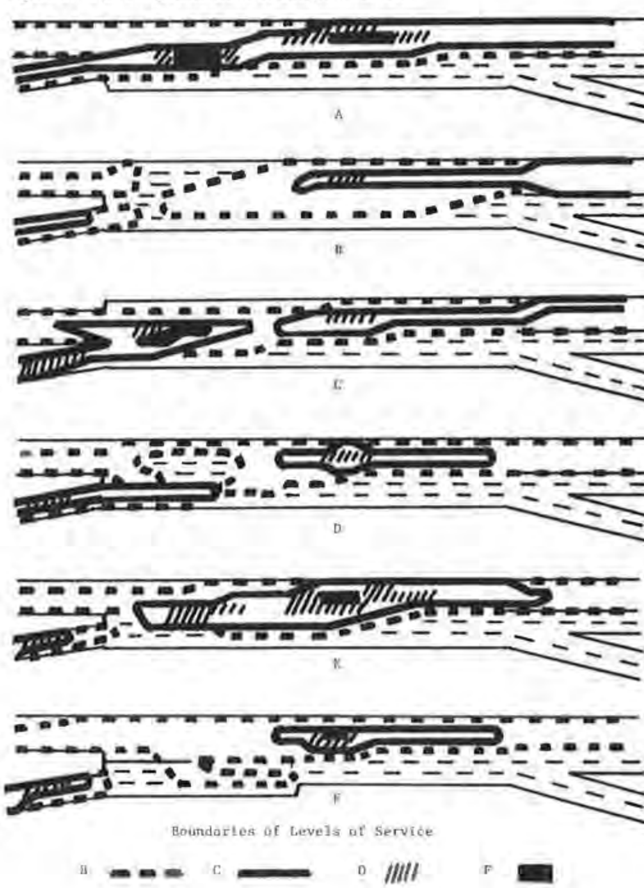
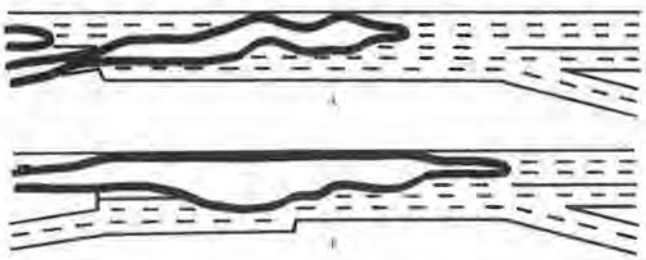


Figure 4. Areas of level-of-service F under oversaturation.



The second condition simulated was the throughput of the weaving section with maximum demands on all lanes that enter the section. The table below shows the results of these analyses in terms of the actual input volumes that could be handled, which indicate the maximum capacity of the weaving section (in vehicles per hour):

Alternative	From Left	From Right	Total
A	5100	3354	8454
B	4947	3686	8633
C	4803	3456	8259
D	5469	3360	8849
E	5163	3774	8937
F	5130	3888	9018
Design	4500	3600	8100

In total throughput, there is a 10 percent difference between the best and worst alternatives. Alternative F allows the highest volumes of 9018 vehicles/h, and these are limited, not by the weaving area, but by the capacity of the three output lanes of I-95. Alternative E gives the best throughput of the six lane options with a capacity of 8937 vehicles/h.

Alternative A has a fairly low throughput capacity of 8454 vehicles/h, and it is of interest that the flow from I-395 of 3354 vehicles/h is lower than the design volume of 3600 vehicles/h. This means that if the flow from I-95 increases more than its design volume, then it will tend to dominate the weaving area and restrict volumes from I-395.

Figure 4 shows the congested areas of the weaving section for the oversaturated condition for alternatives A and F. For alternative A the congestion starts at the weaving and merging areas, which indicates that these areas are the capacity constraints. With alternative F the congestion extends upstream from the far end of the weaving section, which indicates that the constraint is the downstream capacity rather than the weaving area itself.

The indication from these analyses is that conventional weaving analysis can give a design that is not operationally the best that might be available. In this example, it appears that some variation of adding the lane to the middle is the solution that gives the greatest capacity under oversaturation. The center lane appears to facilitate smoother weaving at the start of the section, which is the most critical area. This improved performance occurs despite the fact that most weaving vehicles now have an additional lane to cross, thus increasing the actual number of lane changes.

Although this is only a single example, it does suggest that a better understanding of the weaving process can be obtained from a model that can differentiate traffic conditions both along and across the freeway. The initial merging area is the most important and special consideration should be given to its design. Rather than a tight design that minimizes lane changes, a design that spreads the merging area should have better operational capacity performance.

Other factors that should be kept in mind for the design process are that weaving streams should be separated longitudinally if possible and the design should specifically try to distribute traffic flow uniformly across all the lanes of the weaving section.

CONCLUSIONS

The primary objective of this paper was to show how the essential capabilities of a potentially powerful microsimulation model could be made more generally available. The relevant components of the very large INTRAS model can be reduced to a model that certainly can be accommodated on a desk microcomputer. As such, it should be most suitable for use as a supplemental tool to current macro-analysis methods.

The potential exists in two particular areas: first, for the applied analysis of practical design and operations problems and, second, for research to obtain a more detailed understanding of merging and weaving behavior. In the latter case, the amount of field data needed to further calibrate and validate the simulation is probably much less than that required to calibrate the macro methods in more detail.

ACKNOWLEDGMENT

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REFERENCES

1. Highway Capacity Manual. HRB, Special Rept. 87, 1965.
2. Interim Materials on Highway Capacity. TRB, Circular 212, June 1980.
3. Y. Makigami, L. Woodie, and A.D. May. Bay Area Operations Study--Final Report. Univ. of California, Berkeley, 1970.
4. D. Wicks, B. Andrews, and R. Goldblatt. Development and Testing of INTRAS, A Microscopic Simulation Model, Volumes 1-4. FHWA, 1980.
5. A.G.R. Bullen; University of Pittsburgh. Freeway Microsimulation--Final Report. KLD Associates, Long Island, NY, 1976.
6. A.G.R. Bullen and S.L. Cohen. Car Following and Lane Changing Laws in the INTRAS Model. Presented at the National Meeting of the Institute of Management Science and Operations Research Society of America, Washington, DC, May 1980.

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Application of Time-Series Analysis Techniques to Freeway Incident Detection

SAMIR A. AHMED AND ALLEN R. COOK

An approach for the automatic detection of freeway capacity-reducing incidents based on the time-series analysis techniques formulated by Box and Jenkins is suggested. An autoregressive integrated moving average model of the form ARIMA(0,1,3) has been recently developed to describe the dynamic and stochastic character of freeway traffic variables. This model is used to provide short-term forecasts of traffic occupancies and the associated 95 percent confidence limits. An incident is detected if the observed occupancy value lies outside the confidence limits of the corresponding point forecast. A total of 1692 min of occupancy observations associated with 50 traffic incidents that took place on the Lodge Freeway in Detroit are used in evaluating the algorithm performance. The algorithm detected all 50 incidents. The resulting false-alarm rate is 2.6 percent when constant parameters of the ARIMA model are used and it decreases to 1.4 percent with variable-parameter estimates. Furthermore, the average time lag to detection when constant- and variable-parameter estimates are used is 0.58 min and 0.39 min, respectively.

In a companion paper (1), we noted that there is growing interest in the development of computer-based systems for the surveillance and control of urban freeway traffic. It has also been explained that these systems require short-term forecasts of real-time traffic variables (occupancies and flows) to allow for the operational changes that usually take place after implementing the control decisions. Furthermore, it has been mentioned that when the forecasted value of a key traffic variable is compared with the next observation it can signal a possible change in traffic stream behavior and can suggest a suitable control response. Finally, the stochastic and dynamic character of freeway traffic variables has been modeled by an autoregressive integrated moving average model of the form ARIMA (0,1,3) which follows the analysis techniques described by Box and Jenkins (2).

In this paper, we present a methodology for the automatic detection of freeway capacity-reducing incidents (accidents, stalled vehicles, instances of debris, etc.), based on the developed ARIMA (0,1,3) model. The problem of accurately detecting freeway incidents both in time and space is an integral part of any effective computer-based control and surveillance system. Information that concerns the magnitude of capacity reduction is obviously needed to balance the demand with available capacity by means of route-diversion messages and ramp-metering control. In addition, knowledge of the nature and location of an incident is required to effectively dispatch emergency services and to determine the appropriate kind of help needed at the incident scene.

Several techniques for freeway incident detection have been tried, albeit with mixed success. Of these techniques, computer-based incident detection seems to be the only technique able to detect all incidents and to provide a numerical scaling of the magnitude of capacity reduction--information that could only be guessed at by a human operator. Computer-based incident detection, however, has some inherent limitations. False alarms represent a major operational problem, especially if emergency services are dispatched every time. Another limitation of computer-based incident detection is that it is a blind system, in the sense that it cannot determine the nature of the detected incident (3). Closed-circuit television surveillance, therefore, can be successfully used to supplement computer-based incident detection by providing a human operator with some form of visual validation and assessment of incidents. This can be achieved in a manner similar to that described by Saridis and Lee (4) in their discussion of hierarchically intelligent control and management of traffic systems. In addition, control measures (ramp metering, advisory messages, etc.) can be undertaken immediately without the inevitable time lag where human interpretation--other than confirming the incident--is involved.

ALGORITHM DEVELOPMENT AND EVALUATION

Several algorithms have been proposed for the automatic detection of freeway capacity-reducing incidents and some of them have been in operation for more than two decades. In general, these algorithms can be categorized as pattern-recognition algorithms (5-10) and smoothing algorithms (11-13). The first category of algorithms uses one or more traffic variables to distinguish between incident and incident-free situations. The second category makes use of short-term forecasts of traffic variables to detect the sudden changes in traffic stream behavior associated with incidents. The appealing characteristic of smoothing algorithms is that past trends of traffic variables can predict recurrent congestion while nonrecurrent congestion due to incidents is unpredictable. All of these previously developed algorithms, however, have two major problems: high false-alarm rates and threshold calibration requirements. In fact, the two problems are strongly related because the threshold levels for detection cannot allow for all of the factors that cause variations in traffic flow (time of day, geometrics, pavement and environmental conditions, etc.). Intuitively, the use of real-time estimates that capture the variability in traffic variables as detection thresholds should lessen the false-alarm problem and should improve the overall performance of the detection algorithm.

Motivated by the preceding remarks, the ARIMA algorithm outlined here has been structured so that it includes real-time estimates of the variability in traffic variables. Traffic occupancy has been chosen as the key state variable and an incident is detected if the observed occupancy value lies outside the confidence limits constructed two standard deviations away from the corresponding point forecasts. These confidence limits are given by

$$\hat{X}_{t+1}(\pm) = \hat{X}_t(1) \pm 2\hat{\sigma}_a \tag{1}$$

and

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

where

X_{t+1} = traffic occupancy observed at time $(t+1)$,
$\hat{X}_{t+1}(\pm)$ = approximate 95 percent confidence limits for X_{t+1},
$\hat{X}_t(1)$ = point forecast made at time t,
$e_{t-1}(1)$ = forecast error made at time $(t-1)$,
$\theta_1, \theta_2, \theta_3$ = parameters of a moving average operator of order 3, and
$\hat{\sigma}_a$ = estimate of the standard error of the white-noise variables.

Figure 1. Performance of ARIMA occupancy algorithm during an incident.

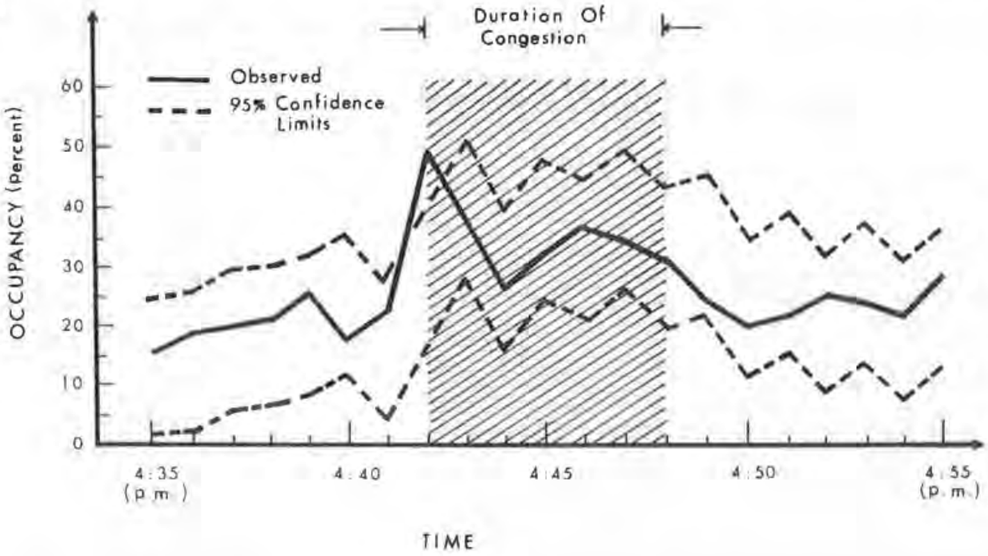
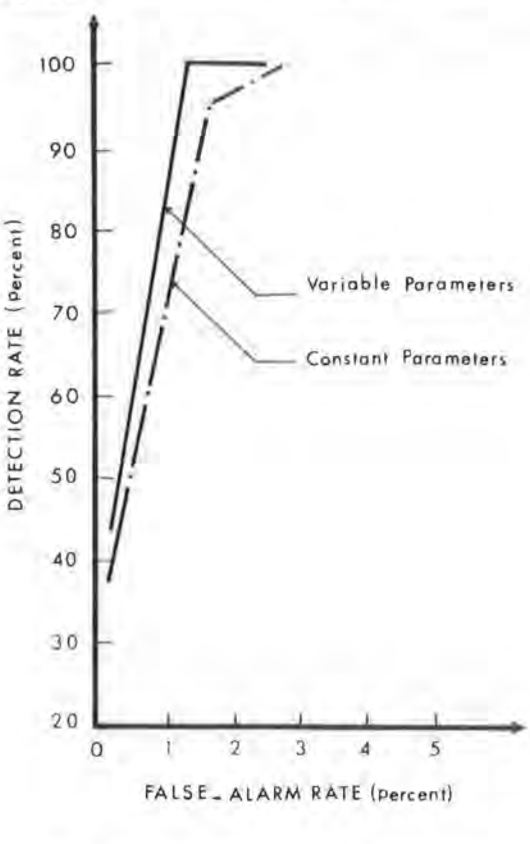


Figure 2. Operating characteristic curves for ARIMA incident-detection algorithm.



To help illustrate, the solid line in Figure 1 represents occupancy observations upstream of a traffic incident that involved a disabled vehicle on the Lodge Freeway in Detroit. The broken lines are the 95 percent confidence limits of the point forecasts. For this particular incident, the algorithm response is indicated by the deviation of the observed occupancy value from the corresponding confidence limits at the onset of congestion development.

Furthermore, it was explained in the companion paper (1) that the moving average parameters of the ARIMA(0,1,3) model vary over time and that it may be desirable to update these parameters occasionally. To explore the effect of this variation on the performance of the ARIMA incident-detection algorithm, a total of 1692 min of occupancy observations associated with 50 freeway traffic incidents were used in the analysis. These incidents took place on a 3.2-km (2-mile) section of the Lodge Freeway and are described in Cook and Cleveland (11).

The confidence limits given by Equations 1 and 2 were computed once by using constant-parameter values estimated from representative incident-free data, and second by using parameters estimated from the observations that cover the incident in hand. The performance results are shown in Figure 2 as operating characteristic curves that relate the detection rate to the false-alarm rate. As noted, the ARIMA algorithm detected all 50 incidents. The resulting false-alarm rate is 2.6 percent when constant-parameter estimates are used and it decreases to 1.4 percent with variable-parameter estimates. Importantly, these are on-line false-alarm rates since they have been computed by using occupancy observations closely associated with traffic incidents. An on-line false-alarm rate is the percentage of false-incident messages out of the total incident messages generated by the algorithm (7).

In addition to detection rate and false-alarm rate, a third important measure of performance is the average time lag to detection. Switching from constant- to variable-parameter estimates reduces the average time lag from 0.58 min to 0.39 min, respectively. Although the detection performance of the ARIMA algorithm improves when the parameter estimates are updated, the question of whether or not to update them must be answered by the operating authority. A trade-off between the benefits gained from improving the algorithm performance and the computational requirements for updating the parameters has to be made.

CONCLUSIONS

The ARIMA incident-detection algorithm described in this paper has shown promising results; however, some further remarks need to be addressed. First, the performance of the algorithm has been evaluated only under heavy and moderate traffic-flow conditions (1200-2000 vehicles/h per lane). More evaluation is required by using incident data recorded during light traffic where the effect of the incident is not remarkably noticeable in the traffic data. Second, work is needed in studying the effect of the data aggregation interval (20, 30, or 60 s) on the detection performance of the algorithm. It is believed that data aggregation induces a masking effect on the actual 0-1 pulses obtained each 0.01-0.25 s from point detectors. Finally, the computational requirements for implementing the ARIMA algorithm by using microprocessors must be examined. As of this writing, the above remarks are being explored.

REFERENCES

1. S.A. Ahmed and A.R. Cook. Analysis of Freeway Traffic Time-Series Data by Using Box-Jenkins Techniques. TRB, Transportation Research Record 722, 1979, pp. 1-9.
2. G.E.P. Box and G.M. Jenkins. Time-Series Analysis: Forecasting and Control. Holden-Day, Inc., San Francisco, CA, 1976.
3. C.L. Dudek. Human Factors Considerations for In-Vehicle Route Guidance. TRB, Transportation Research Record 737, 1980, pp. 104-107.
4. G.N. Saridis and C.S.G. Lee. On Hierarchically Intelligent Control and Management of Traffic Systems. Proc., Engineering Foundation Conference on Research Directions in Computer Control of Urban Traffic Systems, ASCE, New York, 1979, pp. 209-218.
5. K.R. Courage and M. Levin. A Freeway Corridor Surveillance, Information, and Control System. Texas Transportation Institute, Texas A&M Univ., College Station, Res. Rept. 488-8, 1968.
6. R.S. Foote. Development of an Automatic Traffic Monitor and Control System. HRB, Bull. 291, 1961, pp. 79-103.
7. M. Levin and G.M. Krause. Incident Detection--A Bayesian Approach. TRB, Transportation Research Record 682, 1978, pp. 52-58.
8. H.J. Payne and S.C. Tignor. Freeway Incident-Detection Algorithms Based on Decision Trees with States. TRB, Transportation Research Record 682, 1978, pp. 30-37.
9. J.T. West. California Makes Its Move. Traffic Engineering, Vol. 41, No. 4, 1971, pp. 12-18.
10. A.S. Willsky, E.Y. Chow, S.B. Gershwin, C.S. Greene, P.K. Houpt, and A.L. Kurkjian. Dynamic Model-Based Techniques for the Detection of Incidents on Freeways. IEEE Transactions on Automatic Control, Vol. AC-25, No. 3, 1980, pp. 347-360.
11. A.R. Cook and D.E. Cleveland. Detection of Freeway Capacity-Reducing Incidents by Traffic Stream Measurements. TRB, Transportation Research Record 495, 1974, pp. 1-11.
12. C.L. Dudek, C.J. Messer, and N.B. Nuckles. Incident Detection on Urban Freeways. TRB, Transportation Research Record 495, 1974, pp. 104-107.
13. R.H. Whitson, J.H. Buhr, D.R. Drew, and W.R. McCasland. Real-Time Evaluation of Freeway Quality of Traffic Service. HRB, Highway Research Record 289, 1969, pp. 38-50.

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Effects of Rail-Highway Grade Crossings on Highway Users

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Basic research into effects of rail-highway grade crossings on highway users was conducted. The overall objective was to investigate improved techniques for estimating nonaccident effects, such as excess delay, user costs, direct energy consumption, and pollutant emissions. Numerical results also were desired. A microsimulation model is developed for analyzing delays due to train blockages at grade crossings not affected by other highway system bottlenecks. An analytic model is then developed to estimate effects of a vehicle slowing at grade crossings with no train present due to rough surface conditions. These models are validated to the extent possible based on field studies. A sensitivity analysis reveals that for most practical applications, train blockages can be analyzed more easily by using simple equations. A sample application of the method is presented in which 385 grade crossings are evaluated from which design options have been selected. The model developed for analyzing effects of a vehicle slowing with no train present is recommended for further applications, although more extensive validation studies are desirable. Numerical results indicate that nonaccident costs of grade crossings dominate accident costs in the ratio of about 3.5:1. The effects of a vehicle slowing with no train present dominate effects of train blockages in the ratio of about 2:1. The methods developed are felt to represent a significant improvement over earlier techniques for estimating highway-user effects. The methods can be applied to evaluation of alternatives such as rail relocations, construction of grade-separation structures, and crossing-surface improvements. Areas of further research are also identified.

The majority of research on rail-highway grade crossings has been devoted to accident aspects. That such research has been justified is seen in a steady decline in crossing fatalities over time (1). Safety investigators have delved into various areas, which include design, driver behavior, conspicuity, and predictive accident equations, among others.

In contrast, nonaccident aspects of grade crossings have been studied in much less detail. Although major nonaccident aspects have been identified (2, Chapter 3), the research is not well developed. These nonaccident aspects fall into the following categories: delay and increased operating costs for highway users, community barriers (physical and psychological), environmental degradation, incompatible or inappropriate land uses, and increasing operating costs to railroads. The categories are not necessarily separate from one another.

The most important nonaccident aspects, or at least the most readily quantified, are those associated with highway users, such as excess delay, cost, energy consumption, and vehicle emissions attributable to grade crossings. Generally, it is required that some or all of these effects be explicitly considered in plans to alter crossing conditions. Such plans might include schemes for rail relocation, grade separation, or crossing-surface improvement. For proper evaluation, it is desirable to have readily applied techniques for quantifying all or some of these highway-user effects.

In addition to identifying important nonaccident aspects of grade crossings, prior research has proposed methods to analyze individual effects. With respect to highway-user effects, the general shortcoming in these methods is that they are at a gross level. For example, in the estimation of delays and highway-user costs associated with train blockage of a crossing, one major study (3, Chapter 11) assumes highway vehicle and train volumes to be uniformly distributed over a 24-h day. In reality, both vehicle and train traffic can follow greatly different time patterns over a day, which affects delay behavior a good deal. Another difficulty is that blockage times are averaged such that each train is treated as though it causes the same fixed blockage time. Actually, all other things being equal, the expected vehicle delay of one 10-min train is on the order of four 5-min trains.

The other significant source of impact on highway users at grade crossings is the slowing that takes place due to rough surface conditions. The same major study (3, Chapter 11) has recommended the use of fixed average approach speeds and fixed average crossing (minimum) speeds. In reality, vehicle speeds are known to vary around the average in some regular pattern. In the case of a grade crossing, the degree of variation around average crossing (minimum) speed is expected to be greater than at free-flow locations. Such variations should probably be considered in the analysis.

In summary, it is desirable to analyze in greater detail the effects of grade crossings on highway users. The purpose of this paper is to investigate methods of accomplishing this task and to provide numerical results. In this research, there are few preconceived notions of grade-crossing behavior so that various interactions can be studied in detail. Nevertheless, a major goal is to develop practical techniques that can be useful in many applications.

DEFINITIONS

Highway-user effects of grade crossings originate from two major sources: occurrence and nonoccurrence behavior. Occurrence behavior is defined to be the slowing, stopping, idling, and sluggish movement of highway vehicles that takes place when a train is present. Nonoccurrence behavior is defined to be the vehicle slowing at a grade crossing when no train is present due to real or perceived crossing roughness.

It has been suggested that nonoccurrence behavior--a vehicle slowing when no train is present--is not necessarily a cost, since it causes drivers to proceed more cautiously across the danger zone of a grade crossing. Several studies (4-6) have implicitly or explicitly taken this view in studying high-speed rural or sight-obstructed crossings.

Counter to the above is the point that if drivers have to worry about crossing conditions, their attention is diverted from the primary safety task--looking for a train. This view is particularly apropos of crossings with active protection and is embraced in federal highway programs that provide funds to smooth crossings (7). In the absence of comprehensive accident data, and in view of the fact that many crossings of concern have active protection, the assumption in this paper is that forced slowing due to roughness is a highway-user cost.

It is useful also to distinguish between two general types of crossings: isolated and nonisolated grade crossings. Isolated grade crossings are considered to be independent of other traffic system bottlenecks, primarily signalized intersections. Nonisolated crossings are those that cannot be considered independent, perhaps lying near a signalized intersection. As an initial research effort, this paper focuses almost entirely on isolated grade crossings.

FIELD STUDIES

As a first step in studying grade-crossing behavior, field studies were conducted at four crossings located in Hammond, Indiana (population 110 000). Hammond is an excellent area in which to study urban grade crossings; it is the gateway from the East Coast of the United States to the busiest rail hub in the United States, the Chicago Terminal District. More than 185 trains operate through or within Hammond daily, traversing about 110 grade crossings.

The four study sites included three isolated grade crossings and one nonisolated grade crossing. The nonisolated crossing lay immediately adjacent to a signalized intersection in downtown Hammond. The four study sites are as follows: Columbia and the Baltimore and Ohio Chicago Terminal Railroad (Columbia/B&OCT), Kennedy Avenue and Consolidated Rail Corporation (Conrail) (Kennedy/CR), Hohman Avenue and the B&OCT (Hohman/B&OCT), and Sohl Avenue and Conrail (Sohl/CR). Table 1 summarizes the characteristics of the four sites.

Occurrence data were collected by detailed traffic counts both when trains were present and when they were not. Arrival counts (in 20-s intervals) were used to analyze delay and to develop time-of-day profiles of volume. Departure counts (in 10-s intervals) were made after train occurrences to complete the delay data. In this manner, vehicles delayed and time-delay data were gathered and compiled for 26 train occurrences at the four crossings over four days (8).

The arrival data were also analyzed in terms of statistical patterns. It was hypothesized that the arrival pattern might have a significant impact on delay. By and large, arrivals tended to be greater than random, with a few cases of random behavior. Departure counts were similarly analyzed and, as expected, tended to be uniformly distributed.

Nonoccurrence (no train present) field studies were conducted at two of the crossings by using a radar speed gun. Studied were free-flow approach speeds to the crossings and minimum speeds in traversing the crossing. The simple method used was in part predicated on earlier research that used more sophisticated methods (9). Generally, both approach and crossing speeds could be roughly approximated by the normal distribution. These occurrence and nonoccurrence field studies were to serve as the empirical basis of grade-crossing models to be developed.

GRADE-CROSSING MODELS

Occurrence-Delay Model

Although useful, literature on delays at traffic signals (10) is not directly applicable to the case of grade-crossing delays. The literature does suggest problem analysis by using computer simulation. With simulation, detailed vehicle behavior under various crossing conditions can be easily studied.

To treat grade-crossing delays, a microscopic event-scan simulation model has been developed. Written in FORTRAN, the model tracks individual vehicles (automobiles, light trucks, and heavy trucks) to and through grade-crossing blockages, tallying delays and stops along the way.

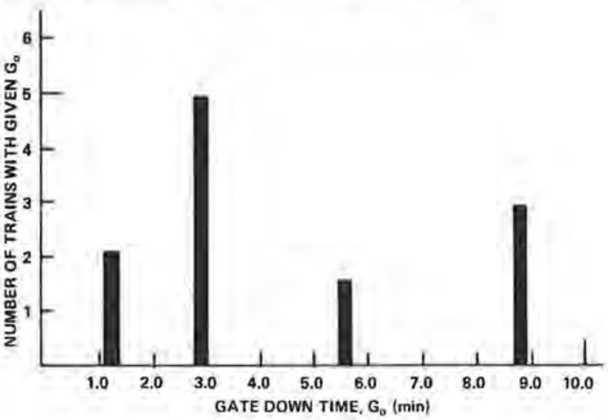
Due to the variable peaking characteristics of highway vehicles versus trains, time-of-day varia-

Table 1. Summary of study site characteristics.

Crossing	1980 Average Daily Traffic (ADT)	No. of Moving Lanes N	No. of Moving Lanes S	Speed (km/h) Avg. Vehicle Approach N	Avg. Vehicle Approach S	Crossing N	Crossing S	Crossing Condition	No. of Railroad Tracks	No. of Trains per Day[a]	Total Gate Time Down (min)	Comment
Columbia/B&OCT	10 500	2	2	48	48	16	16	Very rough	5	45	130	Alternate routes available, but tracks cannot be avoided
Kennedy/CR	18 500	2	2	58	58	37	37	Rough	1	14	40	No alternate routes available
Hohman/B&OCT	10 000	1	1	40	40	29	29	Fairly smooth	2	42	126	Alternate routes available, but tracks cannot be avoided
Sohl/CR	6 500	2	2	40[b]	40[b]	19	23	Very rough	2	28	70	Adjacent to signalized intersection; "escape" route available–grade separation 805 m east

Note: N = northbound, S = southbound.
[a]All freight. [b]Approach speed for vehicles not stopped by traffic signal.

Figure 1. Sample train histogram—one time-of-day period.



tions are explicitly recognized. For highway vehicles, up to 12 time-of-day periods are covered for each travel direction. For trains, up to six time-of-day periods are covered.

Train behavior is treated by a histogram approximation, based on the expected numbers of trains by train blockage time (Figure 1). Train speed is associated with the gate down time of a histogram point. To investigate the impact of increasing train speed, for example, the associated gate down time would be appropriately reduced. The railroads operating over the grade crossings provided the data to formulate the histograms (11).

Arrivals, departures, and speeds of highway vehicles are all treated by Monte Carlo techniques. Vehicle approach speeds to a train occurrence are treated as being normally distributed, while crossing speeds are based on fundamental flow relations.

The results of an occurrence simulation are detailed performance measures summed over individual vehicles, such as total delay time, idling time, speed cycles, and slow-speed time. To ensure statistical accuracy and to identify variability in behavior, each train occurrence is repeated a user-specified number of times and final results are averaged. Performance measures summed over all gate down times in a time period are stored for later conversion into highway-user effects.

After extensive debugging to verify proper processing, the model was run on train occurrences observed in the field studies. The results indicate that the model replicates field delay well as long as good input data are provided.

Nonoccurrence Model

It seems appropriate to characterize nonoccurrence behavior in terms of statistical variations around average speed values. Both approach speed and crossing (minimum) speed would follow regular patterns of variation. By assuming final resumed speed to be identical to approach speed, nonoccurrence behavior can then be treated in terms of speed cycles.

The nonoccurrence model developed follows the principles just outlined. Approach and crossing speeds are assumed to be normally distributed around their mean values. The assumption of final speed equal to approach speed is in part due to effect evaluation data structured in this way (12). Approach and crossing speeds of individual vehicles are assumed to be independent, which is in line with earlier research (9). Last, speed combinations are constrained so that crossing speed is less than or equal to approach speed, as is generally expected.

Time lost in slowing at grade crossings with no train present is not included in this model. The addition of a few seconds travel time to an individual vehicle trip is not felt to merit treatment for value of time (VOT) lost.

This analytic treatment of speed behavior is executed on the computer and applies to all vehicles not affected by train blockages. The results are arrays of speed cycles generated by vehicle type (automobile, light truck, and heavy truck). As with occurrence performance measures, speed-cycle arrays are stored for later conversion into highway-user effects.

Validation of the nonoccurrence model consisted of basic program check-out only. The underlying theory was assumed to be true, since no data base was available for extensive validation.

Effect Evaluation Programs

Once the basic grade-crossing models were developed, effect evaluation programs were written. These programs use the most current data for converting vehicle slowing, stopping, idling, and delay into user costs, direct energy consumption, and pollutant emissions (12;13, Appendix B). In the case of highway-user costs, the operating costs cover the following cost components: fuel, oil, tire wear, vehicle maintenance, and vehicle depreciation.

Table 2. Daily vehicle delay at study sites, 1980.

Crossing	ADT	Time Delay (h)	Vehicles Delayed	Percentage of ADT Delayed
Columbia/B&OCT				
Northbound		26.4	762	
Southbound		19.5	565	
Total	10 500	45.9	1327	13
Kennedy/CR				
Northbound		9.1	353	
Southbound		7.8	315	
Total	18 500	16.9	668	4
Hohman/B&OCT				
Northbound		17.7	530	
Southbound		17.3	519	
Total	10 000	35.0	1049	10
All three crossings	39 000	97.8	3044	8

VOT follows guidelines given in a manual by the American Association of State Highway and Transportation Officials (12). The monetary value is taken to vary by both trip purpose and length of delay and is applied to occurrence delays only. All costs are in 1980 dollars but can be updated at program execution time.

The effect evaluation programs perform straightforward computations, applying the basic effect conversion data to the stored arrays of performance measures. Since the effect conversion data are recorded in convenient tabular form, they can be easily updated as more current information becomes available (e.g., changes in vehicle operating costs with changes in vehicle design).

Program validation consisted of check-out of the computational procedures only. The critical element is the validity of the effect conversion data.

MODEL APPLICATION TO STUDY SITES

With traffic, geometric, and train operating data collected in the field studies plus data available from the Hammond Railroad Relocation and Consolidation Project (11), the three isolated grade-crossing sites were analyzed with the models. Delay, cost, direct energy, and pollutant emission results are discussed in turn.

Delay

Delay results are given in Table 2. Although only daily totals are shown, each crossing was typically analyzed in six time-of-day periods for each travel direction. The maximum volume delayed in any one period and direction analyzed is at Columbia/B&OCT, with about one-quarter of all vehicles delayed in a southbound evening peak period. This is a reasonable result; if the fraction were larger, many drivers would divert to other travel routes.

Overall, most delay is estimated to occur at Columbia/B&OCT, with nearly 46 h of vehicle delay/average weekday, which affects 1300 vehicles out of the average daily traffic (ADT), or 13 percent. This particular crossing is blocked for more than 2 h/day by train activity. In contrast, only 4 percent of the vehicles at Kennedy/CR are delayed for about 17 h/day. The result reflects a much lower level of train activity.

If vehicles delayed were estimated by simply the percentage of time each crossing is blocked times the ADT, the respective numbers of vehicles delayed by crossing would be only 950, 510, and 880--all underestimates. This point illustrates that delay is sensitive to the relative time distributions of highway and train traffic.

Highway-User Costs

The corresponding highway-user costs are presented in Table 3. The added excess cost per day is in the range of \$100-\$300/crossing. On average, the excess is equivalent to drivers having to travel about an additional 0.20 km (0.125 mile) of roadway for each crossing.

Nonoccurrence costs (slowing only with no train present) dominate occurrence costs in the ratio of 2.3:1. This result will be discussed further.

Between occurrence costs, operating costs dominate VOT costs in the ratio of about 2:1. The result partly reflects the nature of the VOT formulation, a point to be further discussed also.

The daily costs can be annualized and are presented in 1980 dollars below:

	Cost (\$000s)			
	Occurrence		Nonoccurrence	
Crossing	Operating	VOT	Operating	Total
Columbia/B&OCT	20	13	56	89
Kennedy/CR	10	4	78	92
Hohman/B&OCT	13	7	18	38
Total	43	24	152	219

Annual costs are further compared with accident costs in Table 4. Without going into detail, accident costs are made up of quantifiable items only, which include property damage, medical costs, public service costs, and other costs (11). There is no assigned value for human pain and suffering, however. It is believed that monetarization of human pain and suffering is pure conjecture and should not be included in a benefit/cost analysis.

Predicted accident costs are based on areawide accident rates. The closeness of the predicted-rate cost and the actual-rate cost is fortuitous. For these three crossings, nonaccident costs dominate accident costs by the ratio of nearly 4:1. If the crossings were perfectly smooth with no slowing costs, the nonaccident and accident costs would be about the same.

Direct Energy

Table 5 presents the direct energy results for automobiles only. Fuel-consumption data for trucks are judged to be lacking in accuracy for use here (12). It is estimated that direct energy would be 10-20 percent higher if truck fuel consumption was included.

The table indicates a relative breakdown between occurrence and nonoccurrence energy about the same as that for cost. Annualizing and converting to barrels-of-oil equivalent (BOE) yields the following:

	Annual BOE		
Crossing	Occurrence	Nonoccurrence	Total
Columbia/B&OCT	245	343	588
Kennedy/CR	105	533	638
Hohman/B&OCT	182	161	343
Total	532	1037	1569

On the whole, these three crossings lead to excess consumption of about 1600 BOE annually, with about one-third due to train occurrences and two-thirds due to vehicle slowing when no train is present.

Pollutant Emissions

Pollutant emissions are also presented in Table 5, summed over both occurrences and nonoccurrences. Included are emissions from all types of vehicles. On an annual basis, these three crossings lead to excess emission of 49 000 kg (54 tons) of carbon monoxide and 7300 kg (8 tons) of hydrocarbons. Nonoccurrence slowing accounts for 75 percent of the excess carbon monoxide and 80 percent of the excess hydrocarbons.

SENSITIVITY ANALYSIS

After completion of initial model runs, the next step was to test the sensitivity of the results to input parameters. An additional goal was to investigate how the study methods might be generalized.

Table 3. Daily highway-user costs at study sites, 1980.

Crossing	Cost (1980 $)			
	Occurrence		Nonoccurence	
	Vehicle Operating	VOT	Vehicle Operating	Total
Columbia/B&OCT				
Northbound	35	22	98	155
Southbound	27	17	74	118
Total	62	39	172	273
Kennedy/CR				
Northbound	17	8	125	150
Southbound	14	4	115	133
Total	31	12	240	283
Hohman/B&OCT				
Northbound	20	11	28	59
Southbound	19	10	27	56
Total	39	21	55	115
All three crossings	132	72	467	671

Table 4. Annual accident versus nonaccident costs at study sites, 1980.

Crossing	Costs (1980 $)				
	Accident		Nonaccident		
	Predicted[a]	Actual[b]	Occurrence	Nonoccurrence	Total
Columbia/B&OCT	19 700	500	33 000	56 000	89 000
Kennedy/CR	18 400	38 400	14 000	78 000	92 000
Hohman/B&OCT	19 700	19 200	20 000	18 000	38 000
All three crossings	57 800	58 100	67 000	152 000	219 000

[a] Predicted accident cost is based on areawide accident rates at more than 350 grade crossings (11).
[b] Accident costs based on actual accidents (1974-1976).

Table 5. Daily direct energy (for automobile traffic only) and emissions at study sites.

Crossing	Direct Energy (kW-h)		Pollutant Emissions (kg)	
	Occurrence	Nonoccurrence	Carbon Monoxide	Hydrocarbon
Columbia/B&OCT				
Northbound	730	1060	27	4
Southbound	560	760	20	3
Total	1290	1820	47	7
Kennedy/CR				
Northbound	290	1400	39	7
Southbound	260	1420	39	6
Total	550	2820	78	13
Hohman/B&OCT				
Northbound	470	440	13	1
Southbound	470	440	13	1
Total	940	880	26	2
All three crossings	2780	5520	151	22

Note: 1 kW-h = 3413 British thermal units (BTUs), and 1 kg = 2.2 lb.

Occurrence Analysis Method

The most important occurrence sensitivity analysis deals with the possibility of simplifying the analysis method. A special study of this was made as follows.

An important finding from initial model runs is that, regardless of other conditions, the average delay per vehicle is constant and is equal to one-half of the blockage time. The most general assumption about vehicle arrivals is that they occur randomly. Last, it is reasonable to assume that vehicle departures from an occurrence are uniformly distributed.

By applying these factors, vehicle delay at train occurrences can be analytically computed by using the Borel-Tanner probability distribution (14). The major result of this analysis is that expected total delay is the same as that obtained by assuming both arrivals and departures to be uniformly distributed.

In the case of the long blockage time usually associated with train occurrences, the random arrival assumption appears particularly appropriate. With this general assumption, occurrence delay can then be easily predicted by the following equations (8):

$$\text{Number of vehicles delayed} = (G_o \cdot q)/(1 - y) \quad (1)$$

$$\text{Total time delay} = (G_o^2 \cdot q)/2 \cdot (1 - y) \quad (2)$$

where

G_o = flow blockage time (gate down time) (min),
q = vehicle arrival rate (vehicles/min), and
y = flow ratio, equal to q divided by the vehicle departure rate with the gates up (saturation-flow rate) (vehicles/min).

These equations can be used to predict delay in most real-world situations. The most important feature of such application is to use as accurate traffic and train data as possible, properly segregated into appropriate time-of-day periods. A following section presents the results of a large-scale analysis on 385 grade crossings by using this full-uniformity model.

The simulation-delay model described earlier should be used where particularly good field data are available that indicate high flow ratios in peak study periods (>0.50) with nonrandom arrivals. In practice, the number of grade crossings fitting these criteria probably is small.

Other Occurrence Studies

Other sensitivity analyses have been conducted by using the simulation model. The first of these indicates that even with nonrandom arrivals, vehicle delay often approximates to the full-uniformity delay of above.

Analysis of variation in delay time over simulation repeats has been conducted. As expected, the total delay coefficient of variation (σ/μ) decreases with increased blockage time. Departure pattern in relation to delay has been studied. The effect is found to be negligible, also as expected.

Some cost effects have also been studied. The major result here is the VOT costs make up a much higher percentage of the total highway-user costs than in the initial runs on the three study sites: 70-80 percent, as opposed to 35 percent. This result is apparently due to the use of a higher percentage truck component plus the nature of the VOT formulation. This formulation sees a step jump in VOT for delays exceeding 5 min. Since this analysis includes two gate down times greater than 5 min, while the earlier analyses did not, VOT costs are higher. The point is that VOT costs must be fully documented to provide meaningful results.

Nonoccurrences

The major nonoccurrence analysis focuses on the use of statistical variation in vehicle speeds as opposed to fixed average approach and crossing speeds. The results indicate that introduction of speed variation around the averages has a significant impact: highway-user costs increase from 5 to 70 percent. The greatest differences occur when average approach speed and average crossing speed are close.

What is also clear from the runs as well is that most of the increased cost is related to the redistribution procedure used in the analysis. This procedure evenly redistributes infeasible combinations of approach and crossing speed so that crossing speed is always less than or equal to approach speed. Based on this study and engineering judgment, it is estimated that the procedure is valid as long as average approach speed exceeds average crossing speed by at least 8 km/h (5 mph).

All of these nonoccurrence studies have also been evaluated with respect to direct energy consumption. Generally speaking, the direct energy impact is smaller than that of costs, averaging about four-fifths the cost variation under like conditions.

Traffic Diversions

An important aspect of grade-crossing behavior not yet discussed is traffic diversions away from train occurrences. In the Hammond area, such diversions were observed frequently in the field studies, which affects delay behavior a good deal. The question then arises as how best to treat this phenomenon in delay analysis.

Unfortunately, there is no easy answer to this question. Short of network modeling with fully calibrated route-assignment algorithms, each crossing must be treated on its own merits. For example, one crossing might lie near an existing parallel route that is grade separated. Drivers might routinely divert to the grade-separated route during train occurrences, which makes estimation of adverse effects of the crossing fairly simple. At the other extreme, another crossing might be miles from the nearest escape route, such that diverting drivers might find themselves worse off than if they just stayed put. In this case, effect estimation becomes complex and difficult to quantify.

The situation with respect to the three study sites considered was much closer to the latter case, i.e., no easy escape routes. In the analyses, no effort was made to adjust occurrence traffic volumes for possible diversions. The delay estimates thus generated were taken as a lower limit on adverse effects of the crossings. Other study sites, no doubt, would merit other treatments.

APPLICATION OF FULL-UNIFORMITY DELAY MODEL

It has been seen that for most practical applications, the full-uniformity delay model should suffice. This model has been used in a major railroad relocation study, as summarized below (11).

The study area covered 385 grade crossings in Lake and Porter Counties in northwest Indiana, which includes the cities of Hammond, Gary, East Chicago, and Whiting. Specifically considered in a preliminary stage were six plans for rail relocation, which were later narrowed down to three. Three analysis years that cover a 20-year planning horizon were studied.

To analyze and compare alternatives, the full-uniformity model was used to estimate highway-user delay, costs, direct energy consumption, and pollutant emissions associated with train occurrences. Nonoccurrence behavior was treated by using fixed average speeds (approach and crossing). This relocation study preceded the research covered in this paper and thus did not include statistical variation in vehicle speeds.

Application of the model included segregation of vehicle and train traffic into separate time-of-day periods, which required compilation of extensive highway traffic and train traffic data from a number of sources. Train behavior was modeled by using the same histogram approach described earlier. The problem of traffic diversions away from congested crossings, which occurred as vehicle volumes grew over time, was treated by manually reassigning traffic to parallel routes. Most of the many occurrence and nonoccurrence computations were carried out in a computer program.

Table 6 summarizes the highway-user effects over all 385 grade crossings for the 1980 existing condition. Estimated accident costs are included in the table.

The estimated 50 million vehicles delayed/year incurred added highway-user costs of about $12.3 million due to the grade crossings. The crossings led to the excess consumption of about 129 million kW-h of energy (76 000 BOE). Excess emissions were estimated to be 2 665 000 kg (3000 tons) of carbon monoxide and 405 000 kg (450 tons) of hydrocarbons. Finally, annual accident costs were placed at $3.6 million, with most of this due to vehicle-train accidents.

Relations between cost components can be compared to those seen earlier. For the 385 grade crossings, nonaccident costs dominated accident costs by the ratio of 3.4:1 versus 3.8:1 for the three study sites. Nonoccurrence costs for the 385 crossings dominated occurrence costs in the ratio of 1.6:1

Table 6. Highway-user effects of 385 grade crossings in northwest Indiana, 1980.

Effect	Total (000s)	Average per Crossing (000s)
Delay		
No. of vehicles	48 950[a]	127
Vehicle hours	1 500	3.9
Cost (1980 $)		
Occurrence		
Operating	2 369	6.2
VOT	2 347	6.1
Total	4 716	12.3
Nonoccurrence	7 582	19.7
Total	12 298	32.0
Energy (kW-h)		
Occurrence	40 554	105.3
Nonoccurrence	88 737	230.5
Total	129 291	335.8
Pollutant emissions (kg)		
Carbon monoxide	2 665	6.9
Hydrocarbons	405	1.1
Accident costs (1980 $)		
Vehicle-train	3 396	8.8
Vehicle-vehicle	188	0.5
Vehicle-property	44	0.1
Total	3 628	9.4

Note: 1 kW-h = 3413 BTUs, and 1 kg = 2.2 lb.

[a]Five percent of total grade-crossing traffic delayed.

versus 2.3:1 for the three study sites. The difference, in part, is probably due to the use of statistical variation in speed behavior for the three study sites, as discussed earlier.

Finally, occurrence costs for the 385 crossings split about evenly between operating and VOT components, versus a ratio of 1.8:1 for the three study sites. The difference may be due to a higher number of delays that exceed 5 min for the 385 crossings, thus leading to higher VOT costs.

Based on the relocation study, the project steering committee has recommended the relocation of 13.5 km (8.4 miles) of a diagonal rail line through Hammond, plus the construction of six grade-separation structures (11). Currently, two of the grade-separation structures are under design and slated for construction in the near future by using federal monies. Preliminary engineering of the rail relocation also will begin soon. Justification for the structures and the relocation rest heavily on the techniques discussed here.

DISCUSSION OF RESULTS

This work has focused attention on interactions and effects of rail-highway grade crossings with respect to highway users. Although initially theoretical, the work has yielded practical techniques. For analyzing train occurrences, the full-uniformity delay model is recommended. For analysis of nonoccurrence slowing, a method based on observed variation in vehicle speeds is recommended.

These methods should find application at a few different levels. On a large scale, they can be used to analyze and compare alternative methods of rail relocation and consolidation. Particularly in this day of increased economic accountability, it is desirable to get as good an evaluation of alternatives as possible. An example application has just been discussed.

On a smaller scale, the methods can help evaluate individual sites considered for improvement. From a local point of view, the insight gained may provide the needed justification to obtain local, state, or federal funds, or may identify where to spend limited funds. From a railroad point of view, the methods might be used to enlist local support for grade-crossing improvements. By recognizing the dollar-and-cent costs of grade crossings, a better spirit of cooperation and communication between railroads and local communities might be achieved.

Although the required computations could be done by hand, it has been found more convenient to computerize the methods. A computer package titled HUGC has been developed, which provides computational ease and ready data manipulation. It has been applied in a few study situations.

Overall, these methods should be combined with techniques for analyzing other aspects of grade crossings and of railroads in general. A more comprehensive framework for first quantifying and then minimizing adverse impacts of railroads can thus be established. The importance of such a framework is becoming clearer every day. It is, for example, in everyone's best interests to minimize the disruption that 100-car coal trains bring to western communities as we attempt to improve the energy situation.

More generally, revitalization of this country's rail system is seen as a major transportation issue as energy and other resource constraints close in on society. It is hoped that the methods discussed here can play some role in the task of expending limited transportation resources where they will do the most good.

FURTHER RESEARCH

From a variety of research needs that surfaced from this work, the following areas are judged to be the most critical. Nonoccurrence behavior at grade crossings has been seen to dominate occurrence behavior frequently. Therefore, more sophisticated speed studies should be conducted than were possible here at crossings fully instrumented with speed traps. Statistical speed distributions, time-of-day variations, and correlations between approach and traverse speed are some of the aspects to be studied.

For occurrence behavior, the question of capacity restraint and route assignment as they relate to traffic diversions needs to be addressed systematically. Another major concern here is treatment of nonisolated grade crossings (i.e., those significantly influenced by other road system bottlenecks such as traffic signals). Computer models for handling the case where a grade crossing and a traffic signal are adjacent to one another have been developed (8). In these models, the separate effects of the traffic signal and the grade crossing are considered. Further research by using these models may be conducted in the future.

The effect evaluations discussed here generally relied on cost, energy, and emissions data that were old, hypothetical, or not well documented. On-going work is needed to keep effect data up-to-date, especially in view of changing vehicle technology.

SUMMARY AND CONCLUSIONS

Due to a lack of prior work, basic research into the nonaccident effects of rail-highway grade crossings on highway users has been conducted. Methods of analysis for isolated grade crossings (i.e., those not affected by other highway system bottlenecks) were first developed to study highway-user effects in detail. The methods consisted of computer simulation and analytic techniques that estimated the following highway-user effects: delay, cost, direct energy consumption, and pollutant emissions. The methods were validated to the extent possible.

Three study sites were analyzed by using the methods, and the total excess effects due to the grade crossings were estimated to be in 1980:

1. Vehicle hours of delay--31 850;
2. Highway-user costs--$219 000;
3. Direct energy consumption (automobiles only)--1580 BOE;
4. Carbon monoxide--49 000 kg (54 tons); and
5. Hydrocarbon emissions--7300 kg (8 tons).

Nonoccurrence costs (vehicle slowing when no train is present) dominated occurrence costs (train present) in the ratio of 2.3:1. Between occurrence costs, vehicle operating cost dominated VOT cost by the ratio of 1.8:1. Nonaccident costs were then compared with accident costs, with the result that nonaccident costs dominated accident costs by the ratio of 3.8:1.

With respect to this last result, it must be emphasized that safety at grade crossings is not readily convertible into dollars and cents. Crossing safety is an aspect that should be evaluated from the outset on its own terms.

Following initial model runs, sensitivity analyses with respect to the methods and parameters were conducted. The first such analysis showed that if the general assumption of random vehicle arrivals could be made, occurrence delay estimation could be greatly simplified. It is recommended that in most real-world studies, a simple full-uniformity model be used. The most important aspect in applying this full-uniformity model is to use as accurate vehicle and train traffic data as possible, being mindful of the different peaking characteristics of the two modes.

Regarding nonoccurrence behavior, the method of treatment originally developed is recommended for further applications. The method treats vehicle speed behavior in terms of statistical variations around average approach and crossing speeds, with crossing speed always less than or equal to approach speed. It is important that accurate speed data be input to the model for good results. Also, it would be desirable to conduct more comprehensive speed studies than were possible here, at grade crossings fully instrumented with speed traps. These speed studies should provide final verification of the recommended method.

One more point is made with respect to nonoccurrence costs. The consistently high value of this effect indicates that installation of a smooth-surface crossing in general is cost effective. The typical $50 000-$100 000 capital investment pays for itself in a few years at many well-traveled crossings. The Federal Highway Administration program in support of such installations has been well-justified [Rail/Highway Crossings (Safety) Program, P.L. 93-87, Section 203].

Traffic diversions away from train occurrences are seen as a basic complication to analysis, with no hard-and-fast solutions. Each crossing must be considered on its own merits.

A sample application of a large-scale analysis was presented by using techniques similar to those recommended here for general use. The application covered alternatives for rail relocation in northwest Indiana, which encompassed more than 380 grade crossings. As a result of the analysis, two grade-separation structures are currently under design and preliminary engineering of rail relocation will start soon.

Based on this research, a computer model has been developed that uses the recommended methods of analysis. The model has been used in a few applications.

Proper evaluation of railroad impacts on their environment is an important transportation task. Evaluation of alternatives is required for a variety of activities, which includes relocation of rail lines, construction of grade-separation structures, and smaller actions such as smoothing of crossing surfaces. This paper has attempted to contribute to an overall framework of evaluation.

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REFERENCES

1. Rail-Highway Crossing Accident/Incident and Inventory Bulletin. U.S. Department of Transportation, No. 1, 1979.
2. A.E. Moon, J. Carter, J. Frank, J.C. Danzig, and B. Whipple. Urban Railroad Relocation: Nature and Magnitude of the Problem. Federal Railroad Administration, Rept. RP-31, Vol. 4, 1975.
3. A.E. Moon, J. Carter, J. Frank, J.C. Danzig, and B. Whipple. Guidebook for Planning to Alleviate Urban Railroad Problems. Federal Railroad Administration, Rept. RP-31, Vol. 3, 1974.
4. T.A. Butcher. Evaluation of Safety Improvements at Highway-Rail Grade Crossings. Purdue Univ., Lafayette, IN, Joint Highway Research Project, Rept. 1, 1973.
5. E.R. Russell. Analysis of Driver Reaction to Warning Devices at a High-Accident Rural Grade Crossing. Purdue Univ., Lafayette, IN, Joint Highway Research Project, Rept. JHRP-74-16, 1974.
6. J.H. Sanders, Jr., G.S. Kolsrud, and W.G. Berger. Human Factors Countermeasures to Improve Highway-Railway Intersection Safety. Federal Railroad Administration, Rept. DOT-HS-800 888, 1973.
7. Railroad-Highway Grade Crossing Handbook. Office of Engineering, Technology Sharing, FHWA, Rept. TS-78-214, 1978.
8. J.L. Powell. Nonaccident Effects of Rail-Highway Grade Crossings by Computer Simulation. Illinois Institute of Technology, Chicago, M.S. thesis, 1980.
9. G.J.S. Wilde, L.J. Cake, and M.B. McCarthy. An Observational Study of Driver Behavior at Signalized Railroad Crossings. Canadian Institute of Guided Ground Transport, Kingston, Ontario, Rept. 75-16, 1975.
10. R.E. Allsop. Delay at a Fixed Time Traffic Signal--Part 1: Theoretical Analysis. Transportation Science, Vol. 6, No. 3, 1972, pp. 260-285.
11. Alfred Benesch and Company. Hammond Railroad Relocation and Consolidation Project, Final Environmental Impact Statement. FHWA, Rept. FHWA-IND-EIS-79-02-F, 1980.
12. A Manual on User Benefit Analysis of Highway and Bus Transit Improvements, 1977. AASHTO, Washington, DC, 1978.
13. Alfred Benesch and Company. Appendices to Hammond Railroad Relocation and Consolidation Project, Final Environmental Impact Statement. FHWA, Rept. FHWA-IND-EIS-79-02-F, 1980.
14. D.L. Gerlough and M.J. Huber. Traffic Flow Theory. TRB, Special Rept. 165, 1975.

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Improving Safety at Passive Crossings with Restricted Sight Distance

JOHN E. TIDWELL, JR., AND JACK B. HUMPHREYS

Investigations were conducted regarding driver knowledge of grade-crossing information, the relation of driver behavior to driver knowledge, and techniques for advisory speed signing. A driver behavior-oriented method for evaluating passive crossings was developed. It was found that most drivers believe that all crossings regularly used by trains have active protection. Driver performance at the sites observed was not related to driver knowledge of grade-crossing facts. It was noted that drivers who looked for trains did not look a proper distance from the crossing or at an appropriate speed to be considered safe. A procedure was developed to assign safe speeds and to locate signing on the approach to the passive grade crossing. Suggestions are made for areas of future investigation.

Drivers who approach a passive grade crossing are expected to take note of the traffic-control devices associated with the crossing and take appropriate action to ensure that they can respond safely to a train near the crossing. One critical point in the system is the amount of sight distance available to the driver for observing trains. The available sight distance must be used by the driver in making a judgment as to whether to stop at the crossing to let a train pass or to continue through the crossing.

The present research included an investigation of driver knowledge of grade-crossing information, observations of driver performance at passive grade crossings with restricted sight distances, and the development of driver behavior-based methodologies for establishing safe approach speeds and evaluating passive crossings. The questions investigated included the following:

1. Are drivers aware of the hazards of grade crossings?
2. Do drivers recognize the standard traffic-control devices associated with grade crossings?
3. Do drivers know their responsibilities at grade crossings?
4. Does improved knowledge of grade-crossing information result in improved performances at grade crossings?
5. How can the actual conditions of the crossing best be communicated to the driver?
6. How can driver behavior at passive crossings be used to evaluate the safety of the crossing?
7. How can passive countermeasures be evaluated?

The present research was divided into four modules, each of which will be reported separately along with the pertinent findings and recommendations. The four modules are as follows:

1. Driver knowledge of grade-crossing information,
2. Driver knowledge related to driver performance,
3. Advisory speeds for passive grade crossings, and
4. Method for evaluating crossings.

DRIVER KNOWLEDGE OF GRADE-CROSSING INFORMATION

Methodology and Results

A 21-item questionnaire was developed to allow an evaluation of driver knowledge regarding grade-crossing-related information. Demographic information was obtained as well as information on exposure to various grade-crossing safety-education efforts. The questionnaire was completed by 829 drivers at a driver's license examining station in Knoxville, Tennessee. Sanders (1) and Dommasch and others (2) have also conducted prior work with questionnaires. However, their responses were obtained in connection with field studies and did not fully cover those items of interest in the present research.

Findings and Recommendations

Significant findings relative to passive crossings were as follows:

1. More than 54 percent of the drivers believe that all crossings or all except those rarely used by trains have active protection,
2. Fifty-six percent of the respondents believed that they were required to stop at passive crossings,
3. Questions that concern passive traffic-control devices were missed by approximately 30 percent of the drivers,
4. Drivers had adequate knowledge of the relative stopping distance of trains and the number of annual fatalities at grade crossings,
5. Exposure to various grade-crossing safety-educational efforts was generally of no advantage in responding to the questionnaire,
6. More than 51 percent of the drivers missed 4 or more of the 11 gradable questions, and
7. Only 4 percent of the respondents indicated that they knew of any enforcement action related to grade crossings.

It was recommended that the following items be emphasized in any public-education effort to improve safety at grade crossings:

1. Only the most hazardous grade crossings have active protection. There are many hazardous passive crossings.
2. The standard traffic-control devices associated with grade crossings should be shown and their placement discussed.
3. Drivers are required to slow down and look and listen for trains at passive grade crossings. A stop is not required except for certain vehicles and at crossings where public authorities have erected a standard stop sign.

It was also recommended that consideration be given to developing unique advance-warning signing to inform drivers that they are approaching a passive crossing. Currently, approach signing and pavement markings are the same for both active and passive crossings, even though vastly different driving behavior is expected. Drivers who approach passive crossings are expected to slow down and look and listen for trains. However, when approaching a crossing with active protection, drivers are expected to maintain speed and carefully observe the railroad-signal devices. The present research demonstrates the low level of knowledge concerning the extent of hazardous passive crossings. Also, the questionnaire responses indicated that enforcement was almost nonexistent as a motivation for safe performance at grade crossings. Therefore, research should be conducted to determine the benefits of enforcement at passive grade crossings.

DRIVER PERFORMANCE RELATED TO DRIVER KNOWLEDGE

Methodology

Two passive grade crossings with restricted sight distance were selected for observing driver behavior on the approaches. An event recorder was used to record time in 100-ft speed traps and to note the instant that the driver made a head movement. By using these data, the vehicle speed profile and the location and speed of the vehicle at the time that the driver looked for a train could be determined. The following dependent variables were determined for each of the drivers observed:

1. Was this a safe driver?
2. Did the driver look for a train?
3. Speed of the vehicle 15 ft from the crossing.
4. Slope of the speed profile approaching the crossing.

The last three digits of the vehicle's license tag were also recorded for later use in correlating with driver knowledge.

The drivers were stopped downstream from the crossing and asked to respond to a four-question multiple-choice questionnaire. The four questions dealt with areas of driver knowledge that were believed to have potential for affecting driver behavior on an approach to a passive grade crossing. These questions dealt with recognition of advance signing and the signing used at the crossing, extent of active protection, and a driver's duty at passive crossings. The portion of the roadways between the grade crossings and the interview sites contained several crossroads. Unfortunately, this caused some vehicles to be observed but not interviewed, and vice versa.

At site 1, the speeds and looking behavior of 94 drivers were recorded. Interview responses from 84 drivers were obtained. Matches of interviews and observations could be made for only 42 drivers.

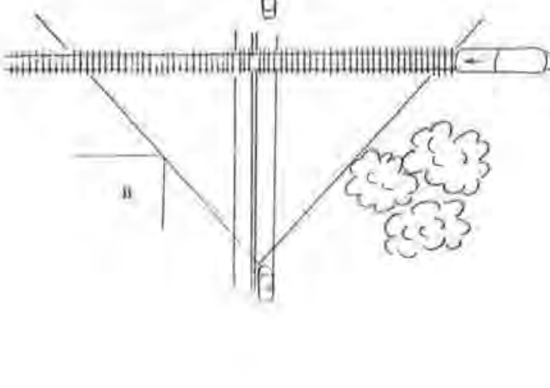
At site 2, the speeds and looking behavior of 122 drivers were recorded. There were 137 drivers interviewed. Matches of interviews and observations could be made for only 47 drivers.

The logic developed by Richards and Bridges (3) was built on developing safe-unsafe criteria for driver-performance evaluation. In order for a driver to have been considered a safe performing driver, two criteria must be met:

1. The driver must look for a train far enough from the crossing to enable a safe stop short of the crossing, commensurate with the vehicle's approach speed, in the event a train were to be detected in the vicinity of the crossing; and
2. Commensurate with the track site distance available from the point where the driver looked, the vehicle's speed, and the maximum expected train speed at the crossing, the vehicle must be able to clear the crossing ahead of a train that might have been barely beyond the available site distance when the driver looked and made a go decision.

If a driver looks for a train and the speed and location of the look are proper, then the driver will safely clear the crossing if no train is detected, or the driver can safely stop if a train is detected. The driver may look many times but, if at least one look meets the safe criteria, then a safe crossing should result. Figure 1 is a schematic drawing of the two possibilities of train arrival relative to a driver's looking behavior. Either of the two conditions may be encountered each time a driver makes a judgment at the crossing.

Figure 1. Two conditions of train conflict as drivers approach crossing and look for trains.



An observed driver was considered to have made a safe judgment if at the time a go or no-go decision process was initiated the vehicle was an adequate distance from the crossing to safely stop (reaction and braking), based on vehicle speed when the judgment was made. Also, vehicle speed must be such that it would allow the vehicle to clear the crossing before the arrival of a train that was barely beyond the sight-distance limits.

In order to compute the total perception and stopping distances required for a safe stop at various speeds, an assumption was made concerning time for drivers to perceive a train at or near a crossing. Prior work, as reported by Richards and Bridges (3), Voorhees (4), the National Cooperative Highway Research Program (NCHRP) (5), and the American Association of State Highway Officials (6), seemed to support the assumption of a 2.5-s perception-reaction time for drivers who approach passive grade crossings or other hazards. Therefore, a 2.5-s perception-reaction distance was assumed in the development of tabulations of perception plus braking distances for vehicles that approach a grade crossing. Since the observed condition of the pavement was dry, it appeared that a reasonable set of assumptions was dry pavement and a 2.5-s perception-brake reaction time. This does not consider panic or emergency action stopping possibilities. A tabulation of stopping distances for various approach speeds was then developed.

The point at which any driver initiated a go or no-go decision process, as manifested by head movements, was plotted on the time-space diagram. By knowing the speed and distance from the crossing when the decision was made, the available track sight distance at the point of the look, the maximum train speed expected at the crossing, and the time it took the vehicle to reach the crossing, the driver could be classified as safe or unsafe.

It had been anticipated that there would be a reasonable proportion of drivers at each site who exhibited safe behavior at these crossings with severely restricted sight distance. However, when the total perception and braking distances required for various approach speeds were compared with the observed values, it was apparent that the majority of drivers was not performing safely. When their approach speed was considered, the drivers looked for trains much too close to the crossing to allow a safe stop short of the crossing. In fact, only the two drivers at site 1 and the three at site 2 who actually stopped could be classified as safe. All of the other drivers were too close to the crossing when their looking movements took place to safely perceive a train and stop before the crossing. All of these drivers did meet the speed criteria for crossing clearance but they were not classified as safe drivers due to their looking too close to the crossing to allow a safe stop. The drivers who looked too close to allow for a safe stop are definitely potential accident victims under some possible situations of train arrival.

To find the relation of knowledge to driver behavior, the 42 matched observations at site 1 and the 47 matched observations at site 2 were grouped by responses to the questionnaire. Those giving correct responses for each question were placed into one group and the remainder placed in the second group. The number of drivers who stopped, number of lookers, mean speed (V) at a distance at 15 ft from the crossing, and the mean speed-profile slope were computed for the two groups for each question at each site. Table 1 shows three of the above measures for the correct and incorrect knowledge groups at sites 1 and 2. No attempt was made to draw a statistical inference from the number or vehicles that stopped (site 1, two; site 2, three) due to the low rates involved.

In order to determine whether the knowledge level of the respondents relative to each of the four questions was related to the dependent variable (i.e., looking behavior of the respondents), a series of chi-square analyses were conducted. This was done for both sites. The data were incorporated into a 2x2 contingency table, correct or incorrect knowledge being a dichotomous variable, as was looking behavior (looked or did not look). Of the analyses completed, only the looking behavior of those correctly answering question 2 (recognition of advance-warning sign) at site 2 were related (0.05 significance level). The findings suggest that those who had more information regarding the advance-warning sign exhibited less looking behavior. This finding was not replicated at site 1. Statistical tests on the pooled data for the dependent variables at the two sites indicated that the data could be pooled. When the data were pooled, the site 2 finding was replicated.

In order to determine whether the knowledge level of the respondents relative to each of the four questions was related to the mean vehicle speed 15 ft from the crossing or the mean speed gradient, a series of Student's t-analyses were conducted. The hypothesis was tested at the 0.05 significance level as to whether the mean speed at 15 ft from the crossing or the mean speed gradient for the correct and incorrect knowledge groupings for each of the four questions at each site was significantly different. These analyses did not yield any significant differences between the two knowledge groups for each of the four questions by using mean speed at 15 ft and mean speed gradient as the dependent variables. This was replicated for both sites and with the pooled data.

The matched responses to the questionnaires given at the two field sites were compared with those obtained through the more detailed questionnaire administered at the driver's license examining station. The short and long questionnaire had four common questions. A comparison was made to see if the knowledge level at the field sites differed significantly from that at the driver's license examining station.

The responses to each question at the three sites were subjected to a chi-square test to see if there was a significant (0.05 level) difference among the responses (correct versus incorrect) of the three groups for each of the four questions (2x3 contingency table). There was no significant difference in the knowledge level for questions 1, 3, and 4 as administered in the field (χ^2 = 2.564, 4.670, and 1.261, respectively; $\chi^2_{0.05}$ = 5.991; df = 2). However, there was a significant difference in the response to question 2 (recognition of advance-warning sign) with the field-site respondents scoring significantly lower on this question (χ^2 = 10.131, $\chi^2_{0.05}$ = 5.991, df = 2). This indicates that the knowledge level of the field sample was lower than the driver's license sample, since the groups rated statistically the same for knowledge on three questions and the field-site responses fell statistically lower for one question. Of course, it is possible that the environment of the questioning may be a factor in the ability of the drivers to answer correctly.

The response to the questionnaire obtained at the two sites were also compared statistically. There was no significant (0.05 level) difference between

Table 1. Driver behavior at sites 1 and 2 and pooled data related to responses to questions.

		Site 1		Site 2		Pooled Data (both sides)	
No.	Question	Correct	Incorrect	Correct	Incorrect	Correct	Incorrect
1	Recognize crossbuck						
	Mean V at 15 ft	29.00	30.75	31.05	36.44	30.15	33.19
	Slope	0.0695	0.0446	0.0455	0.0411	0.0561	0.0431
	Lookers	10	5	19	3	29	8
2	Recognize advance sign						
	Mean V at 15 ft	29.92	29.00	33.52	31.11	31.51	30.28
	Slope	0.0544	0.0614	0.0355	0.0527	0.0461	0.0561
	Lookers	7	8	5[a]	17	12[a]	25
3	Where signals placed						
	Mean V at 15 ft	30.07	29.22	33.37	31.21	31.91	30.24
	Slope	0.0524	0.0574	0.0437	0.0452	0.0475	0.0512
	Lookers	5	10	9	13	14	23
4	Approach passive crossing						
	Mean V at 15 ft	28.75	30.00	30.46	32.71	29.52	31.53
	Slope	0.0669	0.0542	0.0385	0.0454	0.0541	0.0493
	Lookers	5	10	6	16	11	26

Note: V = speed.

[a]Significant (0.05) difference.

the responses to each question at the two field sites.

In summary, both the pooled data and that for each site did not support the hypothesis that good knowledge resulted in statistically better driving performance at grade crossings. The only significant difference between groups occurred with question 2. The group that answered question 2 correctly (advance-warning sign) had a significantly smaller percentage of drivers looking on the approach to the crossing. Therefore, the data did not indicate that the groups of drivers who answered knowledge questions correctly performed significantly better at the sites studied. This generally agrees with findings by Sanders (1) from a related effort.

Findings and Recommendations

1. Needless to say, it was surprising and somewhat disconcerting to learn that only 5 of 89 drivers (all of whom stopped at the crossing) could be classified as safe drivers. Twenty-six drivers did make head movements, which indicated that they were aware of the crossing, but they were not making their head movements when at a safe distance from the crossing or at a safe speed. Therefore, a significant number of drivers looked but still were unsafe due to their looking too close to the crossing.

To assist a looking driver to do so at a proper location and speed, a signing system should be employed at crossings with restricted sight distance that will convey appropriate information to the driver. A standard regulatory speed-zone sign is a candidate countermeasure. A new sign may be appropriate since no current traffic-control device clearly indicates to a driver a safe speed and locates the point where effective looking should take place.

2. The noncorrelation of knowledge with indices of safe performance such as stopping, looking movements, mean speed-profile slope, and mean speed 15 ft from the crossing seems to indicate that variables other than knowledge also have an effect on performance. Of course, a certain base level of knowledge is needed to perform safely. However, possessing that level of knowledge does not guarantee safe performance. (The use of seat belts is a good example of this phenomenon.) This does not negate the need for driver education. If no drivers are informed of the desired behavior, then proper performance will be lower overall compared with that expected if all drivers were properly informed. However, a segment of knowledgeable drivers apparently will still perform unsafely for other reasons, which includes lack of association of grade crossings with hazards.

Further research should be conducted at additional sites to confirm these results. It is hoped that sites might be found that yield more even proportions of safe and unsafe drivers as determined by observed speeds and head movements. The present research did not detect a significant effect. If validated by a future study, the noncorrelation of performance with knowledge may have significant implications on public-information campaigns such as Operation Life Saver.

3. Of the 89 matched questionnaires, 56 (63 percent) of the motorists interviewed believed that they were required to stop at all grade crossings.

Further research should be conducted in this area to determine if the erroneous belief that a stop is required at grade crossings affects driver performance at grade crossings. If the perceived requirement is considered too restrictive, then the driver may simply neglect to perform any special actions as he or she approaches a crossing.

4. The techniques used in this study are workable, but they are limited to sites where observers will not be distracting and where they will also have a view of the speed traps. These criteria severely limit the sites where this technique can be used.

The study technique would be greatly improved if a device such as Federal Highway Administration (FHWA) traffic analyzer was used to develop the speed profile on vehicles that approach the crossing. However, head movement data collection requires hidden observers. If head movements were deleted from the data collection, then a simplified procedure could be developed for research studies and for use by public officials in developing a priority ranking of crossings for the installation of active control devices. The procedure could be used at all crossings, not just those with severe sight-distance restrictions or with the hidden observers. However, the procedure could best be used by diagnostic teams in detailed investigations of selected hazardous crossings.

ADVISORY SPEED SIGNING FOR PASSIVE GRADE CROSSINGS

Methodology

The field observations indicated that drivers at the two sites who desired to perform safely, as manifested by their head movements, actually performed in an unsafe manner. This led to an investigation of a method for advising drivers of the proper speed for approaching a passive grade crossing. Such drivers are expected to drive at an appropriate speed and look and listen for trains. However, the driver is given no information that would assist him or her in selecting the appropriate speed and deciding where to begin looking for trains. The driver needs to have advisory information to offset the lack of information available concerning sight distance and train speeds.

Advisory speed signing is used at hazardous horizontal curves and approaches to intersections with limited sight distance. Lyles (7) recently studied signing systems for the latter, which may have application to the problem at hand. Similar signing for approaches to passive crossings with restricted sight distance would appear to be a low-cost countermeasure to improve safety at passive grade crossings.

In order to establish a safe speed for the approach to a passive crossing with restricted sight distance, the following procedure is suggested:

1. Measure the available right sight distance (RSD) and left sight distance (LSD), as noted in Figure 2. The RSD and LSD should be measured from each of the distances noted in the last column of Table 2. The associated perception-braking distance for the entire range of expected approach speeds should be covered.

2. By using the maximum train speed, determine if a driver who passes a point at the associated speed (for example, 126 ft from the crossing at 20 mph) could clear the crossing if a train was barely out of view when a go decision was made. In other words, based on the distances the train and the highway vehicle would be from the crossing and their speeds, would the highway vehicle clear the crossing ahead of the train? If the highway vehicle could clear the crossing before the arrival of a train barely outside of the sight-distance triangle, then the speed associated with the approach distance under consideration would be a candidate for the

Figure 2. Typical grade crossing for evaluation by using speed procedure.

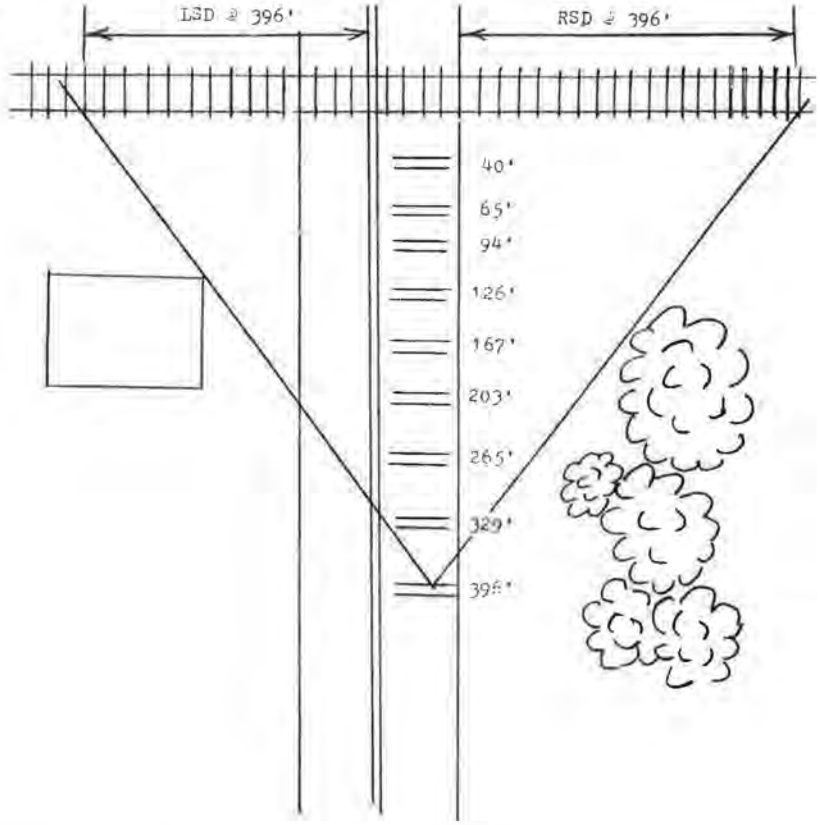


Table 2. Minimum values for use in locating points on approach roadway for evaluation of sight distance.

V (mph)	f (wet)[a]	Perception Distance at 2.5 s (ft)	S[b] (ft)	Required Perception and Braking Distance[c] (ft)
5	0.40	18	2	40
10	0.40	37	8	65
15	0.40	55	19	94
20	0.40	73	33	126
25	0.38	92	55	167
30	0.36	100	83	203
35	0.35	128	117	265
40	0.33	147	162	329
45	0.32	165	211	396
50	0.31	183	269	462
55	0.31	202	325	547
60	0.30	220	401	641
65	0.30	238	470	728
70	0.29	257	564	841

Note: V = speed, f = friction, and S = braking distance.

[a]Coefficient of friction (6).

[b]Based on $S = V^2/[2(f)32.2]$, where V is in feet per second.

[c]General minimum legal clearance for stopped vehicle of 15 ft, plus 5 ft from driver's head to front bumper, i.e., add 20 ft.

advisory speed. For example, assume the track sight distance is measured 126 ft from the crossing. Further, assume that the measurements and computations indicate that a highway vehicle traveling at 20 mph at 126 ft from the crossing can clear the crossing before a train that may be hidden from view at 126 ft from the crossing. In this case 20 mph would be a candidate advisory speed.

3. After evaluating all of the appropriate points from Table 2, the highest candidate advisory speed should be used.

The advisory speed sign should be placed at a location where effective looking can take place. By using this procedure, the driver will be told where to look for a train and the speed at which the highway vehicle should be traveling at the point that looking takes place. If the driver obeys this signing, then the vehicle can either stop short of the crossing if a train is detected or clear the crossing safely if a train were just outside of the sight-distance triangle from the point of the recommended looking. In other words, the driver will be essentially guaranteed a safe crossing if the signing is followed. The site would have to be monitored for changes in the sight distance or train speeds that would affect the advisory speed. The worst-case situation should be the basis for the sight-distance computations.

Findings and Recommendations

Drivers need to know how fast they should be traveling on the approach to a passive crossing with restricted sight distance. They also need to know where looking for trains should take place. The standard advisory speed-warning sign could be used for this purpose. However, this sign generally conveys the message that the posted speed is applicable downstream from the sign. For example, the advisory speed for horizontal curves is located in advance of the point where the lower speed is required. A desirable system would be to post the standard regulatory speed sign at the point where the lowered speed is needed for effective looking. Of course, this should be preceded by advance signing that concerns the reduced speed zone. The standard advance railroad-warning sign should be located between the advance speed-zone sign and the regulatory sign. This would help the driver relate the hazard to the reduced speed zone. Regulatory signing would be useful for law enforcement personnel in judging whether or not a driver was performing safely at a passive crossing. Only 4 percent of the respondents in the Knoxville study knew of enforcement action related to grade crossings. This also might cause the driver to relate possible enforcement action to the crossing. Of course, enforcement action should be a part of the countermeasure. Lyles (7) studied various traffic-control-device treatments in advance of hazardous rural intersections. He found that regulatory signing in conjunction with a standard crossroad-warning sign was more effective than the standard crossroad sign alone. A similar study should be conducted to investigate the effectiveness of regulatory signing at passive crossings. Another alternative would be to develop a new signing system that conveys the following information:

1. The crossing being approached is a passive crossing,
2. The driver should approach the crossing at the speed posted, and
3. The point where effective looking for trains can take place should be identified.

It is recommended that both regulatory signing and new passive advance signing be given careful consideration for future research.

DRIVER BEHAVIOR AS INPUT INTO PRIORITIZATION OF PASSIVE CROSSINGS

Methodology

Sight distance at railroad grade crossings has been used in the past in the evaluation of crossings. Priority and warrant formulas for grade-crossing improvements have also attempted to use sight distance along with other factors as independent variables in the prediction of accidents or in relating the hazardousness of crossings. Also, independent variables used in research efforts to evaluate the effects of countermeasures have attempted to determine if the treatment increased the safety of the system. Here, again, the variables measured were actually surrogates for an evaluation of how the driver was actually coping with the crossing. Did the countermeasure enable or encourage the driver to traverse the crossing in a safer manner when compared with standard treatments or other treatments?

All states have procedures to guide management in determining which passive crossings should be upgraded to active protection. These procedures may be simply a method of setting priorities for the use of available grade-crossing protection funds or they may be actual numerical warrants to be applied to specific situations. There is no nationally recognized formula or warrant for providing active protection. In 1977, Sanford (8) reviewed the criteria used by the states. This review indicated that many of the states were using one or more of the following formulas for setting priorities for grade crossing improvements:

1. Hazard formula from NCHRP Report 50 (5) (used by 10 states),
2. The New Hampshire formula (4) (used by 7 states), and
3. The Peabody-Dimmick formula (9) (used by 6 states).

These formulas consider only the volume of highway and train traffic in conjunction with the present protection being provided. It should be noted that sight distance is not an input into the hazard index formula. The NCHRP report (5) procedure does allow adjustments for approach gradient number of lanes and angle of crossing. Sanford (8) also reported that 15 states and the District of Columbia used view and site conditions as parameters in priority formulas and/or in warrant formulas. These parameters are based on the judgment of the jurisdiction as to how important sight distance is to total safety. Sight distance is used subjectively by these states or as a weighted factor in an overall equation that considers several other variables.

None of the procedures currently available for use in evaluating countermeasures or for establishing priorities in a program for upgrading passive grade crossings actually consider driver behavior at the passive sites. Various measures are used as input into models that endeavor to rank the probability of train-vehicle collisions during a certain time period. However, current driver behavior at the crossings under consideration is a meaningful independent variable that could be used in establishing priorities.

The probability of a train-vehicle collision is related to the exposure of unsafe drivers to trains. Unsafe drivers are those who operate their vehicles in such a manner that they could not avoid an accident if a train were in the vicinity of the crossing while they approach that crossing. The probability of conflict is related to the probability of train arrival during the time that a driver is approaching the crossing in an unsafe manner. In order to approximate the probability of this conflict, an estimation of the number of potentially unsafe drivers who use the approach to the crossing under consideration must be determined.

As discussed earlier, the design of the field studies for research into driver behavior resulted in the concept of a window of speeds at each point along an approach roadway where a safe go or no-go decision could be made by the driver. The current literature indicates that present sight-distance measurement procedures are based on measurements of track sight distance taken at one or two locations along the approach roadway. However, the very nature of sight obstructions and the possibility of vehicles approaching the crossing at varying speeds can allow good sight distance at one point on the highway and poor sight distance at a point within the next 10 ft. Therefore, measurement from a standard point in the highway does not provide a complete measure of the influence of sight distance on the safety of the crossing. Sight distance is site specific as related to safety and the establishment of advisory speeds.

As discussed above, methods used by the states to evaluate grade crossings for priority ratings or warrants either overlook sight distance or include a judgment or factor adjustment to the ratings. The sight-distance factors used by the states are empirical factors based on the judgment of the developers of the formulas.

The procedure that follows has the potential of providing a direct index of the effect of sight distance on the safety of a crossing. The procedure consists of the following:

1. Measure the track sight distance from many locations on the two highway approaches.
2. By using the FHWA traffic analyzer or a similar technique, measure the speed of each vehicle as it crosses each point where track sight distances were determined.
3. Evaluate the speed and sight-distance data and determine the proportion of drivers who at some point on their approach to the crossing were operating at a speed where they could make a safe go decision (could stop safely and also clear the

crossing before a train just outside the sight-distance triangle would arrive).

The evaluation of speed data would be based on the criteria that (a) if at any one of the sample points the vehicle was traveling at a safe speed, then the driver is labeled a potentially safe driver; and (b) if the speeds at all evaluation points are outside of the safe window at each point, then the driver is labeled an unsafe driver.

The proportion of unsafe drivers could be used as input into a priority procedure by factoring the average daily traffic (ADT) to arrive at the expected number of drivers who are using the crossing in an unsafe manner. The proportion of safe drivers could also be used to evaluate countermeasures through before-and-after measurements. While this technique may be too time consuming for use at all passive crossings, it certainly appears that it would be useful to diagnostic teams as they study specific crossings. The technique will reveal how drivers perceive the hazardousness of the crossing.

The basic procedure can be used at all crossings since no hidden observers are necessary. It can also be used at night since head movements are not recorded. It could be used on a sampling basis for each approach, for use in expanding to 24-h traffic. The data collected would give direct consideration to measures currently being used in empirical formulas by the states. Such items as vehicle speed, train speed, crossing angle, highway alignment, and approach grades are considered directly by the driver along with sight distance as the driver drives through the crossing. Vehicle speed at the various points indicates how safely the driver is using the crossing. The real questions are, Could the driver stop if a train suddenly appeared? and, also, Could the vehicle clear the crossing after making a go decision? These would be a direct evaluation of a crossing's safety. If two crossings have the same car-train exposure, the one that is being operated with a higher product of unsafe drivers and trains should be given higher priority for grade-crossing improvement. The concept of dealing with unsafe drivers could have application to any ranking process by deleting the factors that are being evaluated directly, such as speeds, sight distance, etc.

A possible objection to the use of this procedure is that the assumption must be made that the driver is always alert and watchful for a train if the driver is labeled safe because the vehicle was traveling at the appropriate speed. This objection is removed if the safe driver is viewed as a potentially safe driver. In other words, if this procedure indicated that 50 percent of the drivers who use a crossing were traveling at an appropriate speed at some point on the approach to allow a safe go decision, then 50 percent would be the absolute upper limit of safe drivers. It may be that the attentive drivers who had proper speeds would be less than 50 percent, but one could be certain that 50 percent were unsafe since they could not have made a safe judgment due to their speeds. This would allow the procedure to be used for comparative purposes, assuming that the matter of interest is the relative number of unsafe drivers for input into countermeasure evaluation or priority procedures.

The real advantage of this procedure is that a human-factors-type measure of how a driver evaluates a crossing is obtained. If the driver evaluates it incorrectly, the speed profile will be an indicator of the wrong evaluation. If the drivers are not perceiving the restricted sight distance or train speed, the use of advisory speed signing should be considered and the site restudied. If the site continues to remain high in the priority listings, the installation of active protection or other countermeasures should definitely be considered.

The recommended procedure, then, is to develop for the approaches of each crossing under consideration an estimation of the number of drivers who approach the crossing in an unsafe manner each day. A train-unsafe vehicle product can then be used to establish priorities. A passive site with 10 trains/day and 400 vehicles that approach the crossing in an unsafe manner (both approaches) would have an exposure of 4000 train-unsafe vehicles. If this were compared with a passive site with 8 trains and 600 unsafe drivers, the exposure of 4800 would indicate that the second site should be considered for active protection first.

A comparison of this procedure with some of the currently used priority procedures was conducted with assumed data for a group of crossings. This comparison yielded a vastly different priority ranking of the crossings, which indicates the great influence that actual driver behavior can have on the relative safety of a group of crossings (10).

Findings and Recommendations

The procedure described can assist diagnostic teams by yielding a human factors evaluation of crossings under consideration. This procedure will also yield measures for use in countermeasure research.

This procedure should be considered by public authorities responsible for managing a grade-crossing improvement program. The procedure should be particularly helpful to diagnostic teams as they evaluate specific hazardous crossings and develop recommendations. Countermeasure research at passive crossings should also consider using the procedure.

ACKNOWLEDGMENT

The research reported here was conducted as part of a larger research effort in conjunction with John E. Tidwell's dissertation (10). The procedures, base data, and detailed findings are reported therein.

REFERENCES

1. J.H. Sanders. Human Factors Countermeasures to Improve Highway-Railway Intersection Safety. Federal Railroad Administration and National Highway Traffic Safety Administration, U.S. Department of Transportation, July 1, 1979.
2. I.N. Dommasch, R.L. Hollinger, and E.F. Reilly. Passive Control at Rail-Highway Crossings. New Jersey Department of Transportation, Trenton, May 1977.
3. H.A. Richards and G.S. Bridges. Railroad Grade Crossings. In Traffic Control and Roadway Elements--Their Relation to Highway Safety, Highway Safety Foundation, Washington, DC, 1968.
4. A. Voorhees. Factors Influencing Safety at Highway Rail Grade Crossings. NCHRP, Project 3-8, Jan. 1967.
5. Factors Influencing Safety of Highway-Railroad Crossings. NCHRP, Rept. 50, 1968.
6. A Policy on Geometric Design of Rural Highways. American Association of State Highway Officials, Washington, DC, 1965.
7. R.W. Lyles. Evaluation of Signs for Hazardous Rural Intersections. TRB, Transportation Research Record 782, 1980, pp. 22-30.
8. J.L. Sanford. Criteria Used by State Highway Agencies to Determine Warrants and Priorities for Warning Devices at Rail Highway Crossings. Highway Traffic Center, Urbana, IL, April 1977.
9. L.E. Peabody and T.B. Dimmick. Accident Hazards

at Grade Crossings. Public Roads Magazine, Washington, DC, Vol. 22, No. 6, 1941.
10. J.E. Tidwell, Jr. Highway-Railway Grade Crossing Safety: Driver Perception of Hazard and Risk. Univ. of Tennessee, Knoxville, Ph.D dissertation, Dec. 1980.

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Radar-Platoon Technique for Efficient and Complete Speed Measurements

SAM YAGAR AND MICHEL VAN AERDE

A technique by which a single observer can accurately estimate speeds of all vehicles in a single lane of traffic with the use of a radar device, regardless of the volume of traffic in that lane, is described. This is accomplished by considering platoons of vehicles rather than each individual vehicle. For each platoon, the speed, platoon composition, and lead-vehicle type are recorded. The speed is measured by means of a radar unit while the platoon composition and platoon lead-vehicle type are observed visually and recorded manually. On two-lane highways, this technique can provide detailed volume counts and speed measurements for one direction of traffic flow and summarized (e.g., 5-min) vehicle counts for the opposite direction. For total two-way volumes of less than 500 vehicles/h, the single observer can provide detailed volume counts and exact speed measurements for both directions.

Although there exist many direct and indirect methods for measuring the traffic characteristics of a highway, they tend to be geared toward one specific type of measurement and do not produce, by themselves, the type of complete data that are required for a thorough analysis of highway performance. Any extra data must therefore be obtained by using other techniques and additional resources. Some of these basic data-acquisition methods are listed below and described in a previous paper (1):

1. Volume counts--visual observation, mechanical counter, and microcomputer; and
2. Speed measurements--license-plate matching, stopwatch technique, microcomputer, and radar.

There are many indicators that either individually or collectively indicate how effectively a highway accommodates various levels of traffic. The total number of vehicles and the average speed of these vehicles are two of the most commonly obtained statistics. Although speed and total volume are perhaps the most important indicators of the operating performance of a highway, other types of data are needed for a more complete analysis.

It is often important to know the vehicle composition of the traffic stream, for one can seldom consider either a truck, a bus, or a motorcycle to be equivalent to a car in this regard. In addition, one is also interested in the spatial distribution of these vehicles, for again it is important to know if these vehicles are all traveling as a single group, known as a platoon, or as individual, independent units.

Similarly, one needs to know more than just the average speed of all vehicles. To know the frequency distribution of their speeds and to determine to what degree drivers are prevented from driving at their desired speeds are often of equal or even greater importance.

Obtaining this type of complete data has been expensive and time consuming. One could either employ excessive amounts of resources or settle for a smaller data set.

With this thought in mind, the radar-platoon technique was developed and tested (2). After 500 h of application in the field it has been found to be fast, accurate, and relatively inexpensive in terms of time and money. These advantages make this method for monitoring the performance of a highway most efficient and highly practical.

DESCRIPTION OF RADAR-PLATOON TECHNIQUE

The radar-platoon technique for obtaining traffic data is based on the use of a radar unit for measuring speeds and the division of the traffic flow into platoons for the purpose of assigning these speeds. Whereas various other techniques for measuring speeds require either extra calculations or several people to take a single reading, the radar unit operated by a single individual automatically produces instantaneous values of speed.

Even with the radar unit, it is not always possible to individually record the speed of every vehicle that passes by, especially at higher volumes. However, vehicles tend to form groups (platoons) at these higher volumes. Since platoons travel as one unit, the average speed of the platoon can be taken to adequately represent the speed of each member of that platoon for most practical purposes. The division of the traffic flow into platoons therefore allows a representative speed of each vehicle to be recorded.

The radar-platoon technique consists of counting the number of vehicles of each type in each platoon, recording the platoon lead-vehicle type, and recording an average speed of the given platoon from the radar unit. This can be accomplished without great difficulty by a single person, as the speeds of vehicles in a platoon are virtually identical.

All data collection for a given location is carried out by one person positioned along the road. The data-collection equipment should be sufficiently removed from the lane in which the traffic is moving (about 5 m or more, if possible) so that it does not affect traffic and preferably on the same side of the road as the principal-flow lane. Although the equipment usually consists of a vehicle that houses the radar set and operator, it is preferable to have the operator completely off the roadway, with only the radar antenna at the side of the road and camouflaged in some manner.

The use of the radar-platoon technique is described in terms of two-lane highways where it has been principally used to date. From the selected

Figure 1. Typical roadside radar setup.

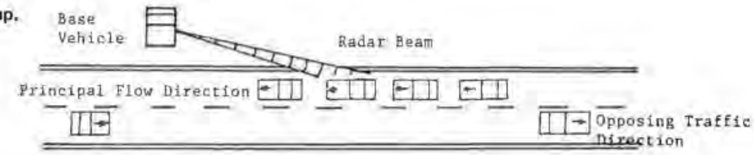
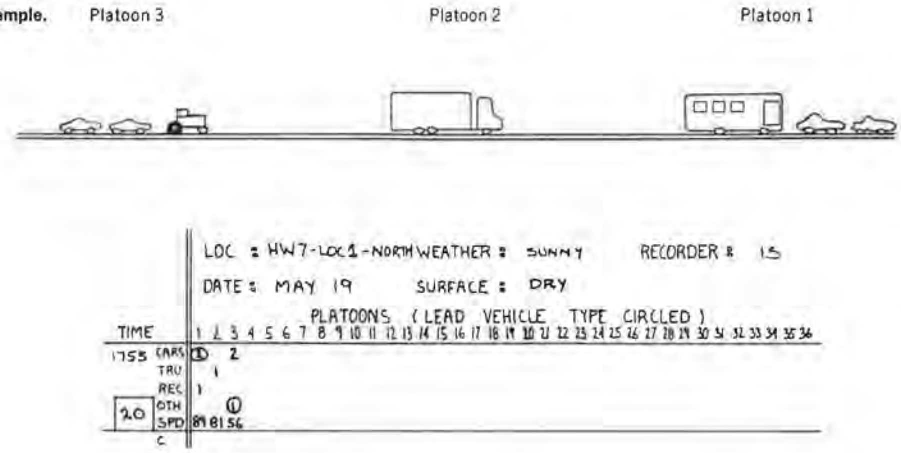


Figure 2. Data-recording example.



site the radar is trained on the oncoming traffic that is moving in the principal-flow direction. Positioning the vehicle and setting up the radar unit take approximately 10 min. The person is then ready to begin collecting data from either inside or outside the base vehicle. A typical geometric configuration is illustrated in Figure 1.

As discussed above, under certain terrain conditions, one may wish to collect data without having to park a vehicle on or adjacent to the roadway. In that case the power supply for the radar unit can be a portable car battery. In either case, the radar power drain is sufficiently small to allow one to collect data for at least 18 h before recharging the battery.

Obtaining Data for Principal-Flow Direction

The study period is divided into a series of consecutive time periods (5 min is recommended), during which traffic characteristics can be considered as effectively time stationary. All data collected during this 5-min period become a single record of information.

The observer considers platoons of vehicles that are observed to be traveling together. These platoons can be easily identified as a group of vehicles that are traveling close together at a common speed, i.e., following a lead vehicle that restricts the speed of any followers.

As each such platoon moves by in the principal-flow direction, the number of vehicles of each type (i.e., cars, trucks, recreational vehicles, etc.) are counted and the type of vehicle that leads the platoon is noted. Also, a single speed can be recorded that is representative of the speeds of all vehicles in the platoon. The fact that there is only one speed for each platoon is part of our definition of a platoon and makes it possible for a single person to record all of this information. If the observer feels that one vehicle is overtaking another or falling behind, he or she need not treat them as being in the same platoon. The platoons are later aggregated during the data-processing phase to obtain any summaries required for the 5-min period, such as total volume, average speed, etc.

Figure 2 illustrates a series of three platoons traveling at speeds of 89, 81, and 56 km/h, respectively, and the recording of these platoon characteristics on a data sheet.

Data on Opposing Traffic Flow

Simultaneous to the data collection for the principal-flow direction, the opposing traffic volume is counted on a hand-held counter. For the opposing direction, no distinction is made with respect to either platoon size, platoon lead-vehicle type, or the type of vehicles in the platoon. At the end of the 5-min period, the accumulated total opposing traffic volume is recorded and a new period begins where the previous one left off.

Data Sheet

A typical data sheet, which has been used in Ontario, is illustrated in Figure 3. This sheet, which is approximately 21x36cm, has been reduced for recording the data-collection site, date, weather, and surface conditions as well as the name of the recorder. On the data sheet in Figure 3 there is room for a total of 36 platoons in each 5-min period. If this number is not exceeded for any 5-min period, data for a full hour (12 5-min periods) can be recorded on one sheet.

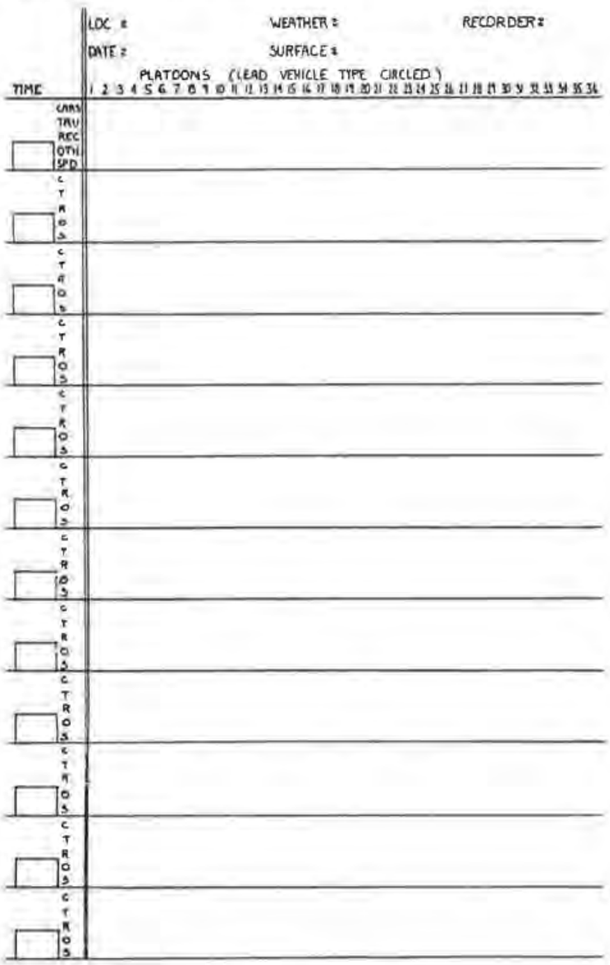
As stated previously, data are collected for consecutive sets of 5-min periods. On the data sheet a five-line record has been reserved for each such time period. A single vertical column of five squares represents the relevant characteristics of a single platoon, as shown in Figure 2. The number of cars, trucks, recreational vehicles, and other vehicles and platoon speed is recorded in that order.

The platoon lead vehicle is recorded by using the convention that the lead vehicle of each platoon is circled unless there is only one vehicle type, in which case the single vehicle type must be the lead vehicle by default.

TYPES OF INFORMATION PROVIDED BY RADAR-PLATOON TECHNIQUE

The radar-platoon technique yields a complete data

Figure 3. Data-collection sheet.



set, which is described below in the various categories.

Detailed Volume Counts

The procedure yields total volume counts for each direction and a breakdown into the various types of vehicles for the principal-flow direction. A total vehicle count is usually used as a measure of road use. It is obtained by summing the number of vehicles of all four types over the given time period.

A more representative measure of road use is obtained when the heavier and longer vehicles, such as trucks or recreational vehicles, are weighted more heavily than standard passenger cars. A classification count can more accurately represent traffic conditions as larger vehicles take up more space, are more difficult to overtake, and cause greater deterioration of the road surface.

Detailed volume counts are more useful than simple aggregated totals, as they can be more fully analyzed and manipulated in the analysis stage of a study. The volume of opposing traffic is useful as it can be a critical factor that restricts the passing maneuvers of the vehicles traveling in the principal-flow direction. Passing, or the lack of it, can break up or cause platoons, respectively. Such platoon formation behind slow moving vehicles has a direct bearing on speeds and quality of service.

One-Way Speeds

The radar method of measuring speeds yields accurate data on each vehicle traveling in the principal-flow direction. This method of data collection is therefore very efficient and, the fact that a 100 percent speed sample is obtained, makes the procedure unbiased either toward or against a particular type of vehicle.

Another major advantage of using radar is that it yields spot speeds as opposed to space average speeds. Other methods of collecting speed data rely on measuring the length of time that a vehicle requires to travel a given distance. The ratio of the distance over time is then used to yield a space average speed for the entire section. However, different factors influence traffic to varying degrees as one travels along any finite section of highway. When using a space average speed approach, these factors are therefore difficult to isolate. In contrast, the spot method yields instantaneous values of speed. These instantaneous values of speed are linked more closely to any set of influencing factors so that their specific impacts can be analyzed.

Two-Way Speed Measurements

For high traffic volumes, detailed counts and speed measurements are taken only for the principal-flow direction. However, when the combined volume for both directions is less than 500 vehicles/h, the radar set can be trained to track vehicles in both directions. An experienced data collector can then obtain speeds, count the individual types of vehicles, and record the platoon lead vehicles for both the principal flow and the opposing traffic. The simultaneous analysis of two directions significantly improves the efficiency of the data-collection process. Also, this type of data can have a higher information content by providing more detailed information on opposing flow for each direction.

At higher volumes it becomes very difficult to visually monitor the data for both directions simultaneously. There is too much work for a single observer, and the principal flow of traffic also eclipses the radar beam, which makes it very difficult, if not impossible, to use the radar for the opposite direction. Complete simultaneous traffic analysis of two directions by a single recorder is therefore restricted to low volumes by both human and mechanical factors.

Platoon Measurements

The behavior of individual drivers can be highly dependent on other users of the highway, especially at higher volumes. A driver is generally influenced by, and sometimes completely restrained by, the vehicles that are either ahead or behind, as well as the oncoming traffic. It is therefore important to study both the behavior of the platoon and the behavior of the individual.

The radar-platoon technique yields directly the type of information required for this type of consideration. Each record contains the size, speed, composition, as well as the leader of each platoon.

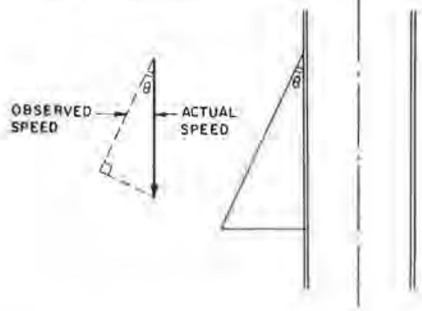
EQUIPMENT CONSIDERATIONS

In setting up, describing, and using the equipment, a number of factors should be considered. These are described below.

Positioning of Equipment

The selection of a site for positioning the radar

Figure 4. Geometric factor for measurement at an angle.



set is determined by two opposing factors. The site that is chosen should therefore be a compromise between these factors. It is very important that the data-collection process not interfere with or affect the natural, normal traffic flow by creating curiosity or fear of a speed trap. If this does occur, speeds may be artificially low and of questionable value in any statistical analysis.

In order to avoid interference problems, the radar set should be well hidden and the observation vehicle should be parked as far off the road as possible. Unfortunately, as one moves away from the road, the angle between the road and the radar beam becomes larger. While small angles produce only insignificant errors, the use of larger angles results in a considerable underestimation of the speed of the vehicle. The geometric factor is illustrated in Figure 4. The calculations and table below can aid in the selection of the most appropriate distance from the road for the radar set:

Measured = actual x cos θ.

Error = actual - measured = actual (1 - cos θ).

Percentage error = (error/actual) x 100 percent
= [actual (1 - cos θ) x 100 percent]/actual
= (1 - cos θ) x 100 percent.

Angle (°)	Error (%)
0	0
2.5	0.10
5	0.38
7.5	0.86
10	1.52
15	3.41
20	6.03

Calibration of Equipment

The utility of the collected data can be no better than the accuracy of the data; it is therefore essential that the radar equipment that is being used be in good operating condition. The radar unit is equipped with an internal calibration circuit. This circuit should be used to check the operation of the internal parts at least once every hour. Each set up of the microphone or antenna should be checked by means of a set of tuning forks. No further data should be obtained once the radar unit fails either test, as such data would be invalid.

During the data-collection process the persons operating the radar set should always ensure that the values of the digital readout are compatible with visual estimates. Any inconsistencies should be noted along with the data that may be affected.

OTHER CONSIDERATIONS IN USING RADAR-PLATOON TECHNIQUE

Some factors that one should consider in designing a data-collection technique and recording form are described below.

Weather and Surface Factors

Weather and surface conditions are important factors that influence the speeds of the drivers and the capacity of the highway. It is therefore essential that the prevailing weather and surface conditions be carefully recorded on each data sheet. Although the weather and surface conditions can be described in great detail by using several adjectives, the use of the computer restricts this vocabulary to a set of letter codes, as shown below:

1. Type of vehicle (leader to be marked by a circle)--C, car; T, truck; R, recreational; and O, other.
2. Weather--S, sunny; O, overcast; R, rain; F, freezing rain; S, snow; D, drizzle; and C, clear.
3. Surface--D, dry; W, wet; I, ice; S, snow covered; C, center bare; L, slush; and A, damp.
4. Location code--highway number, 1-9 (e.g., highway 8, northbound, location 2); direction, N, W, S, and E; and location, 1-9 (e.g., 8 N2).
5. Date--month, day, hour, and minute (e.g., July 20, 7:35 p.m. = 07201935).
6. Recorder--identified by initial of surname or other single-digit code.

In general, weather and surface conditions should be updated every 5 min (if changes occur). When a dramatic change takes place during a 5-min cycle, the conditions that were dominant are assigned to the entire period or, alternatively, that period can be eliminated. Very sudden and drastic changes or exceptional circumstances should be noted alongside the affected data.

Cycle Length Selection

A counting cycle of 5 min is recommended because either a longer or a shorter cycle would lead to increased difficulties. In addition, 5 min is a convenient time span for calculations and physical checks.

The radar-platoon technique is based on the fact that speeds can be accurately assigned to each platoon that passes the observation site during the timing cycle. During a 5-min cycle, an average of approximately 20 platoons will go by. This means that if the last platoon in each 5-min cycle runs over into the next time period, at the most 5 percent of all the platoons will need to be broken up between the adjacent two time periods. However, this percentage doubles if the time period is cut in half and becomes 25 percent when the cycle length is 1 min.

The above argument favors long counting cycles. However, as cycle length increases, the data become more aggregated and special conditions that prevail for only a short time become hidden in the remainder of the data with which they are averaged.

For those two reasons a time length of 5 min was chosen as being short enough to show any short-term fluctuations and yet long enough to keep the number of split-up platoons to an acceptable level.

Platoon Measurement

A platoon is a physical group of vehicles whose motions are interdependent. The speed of a platoon is dictated by the lead vehicle. The speeds of the following vehicles might fluctuate slightly but their average corresponds approximately to that of the leader. This average speed is assumed to be representative of the whole platoon and is recorded on the data sheets. It is quite simple to mentally integrate and average the speed of a platoon as the speeds do not fluctuate significantly, and the recorder is provided with a relatively long time to record the platoon speed if the platoon is long.

Even when the number of broken-off platoons is kept to 5 percent by the use of a 5-min cycle, as described in the previous section, it is important that both the recorder and the person who will eventually analyze the data have the same understanding as to how this 5 percent will be treated. Furthermore, this convention is intended to treat these split-up platoons in such a way that in the final analysis they will represent the actual events as close as possible.

Small platoons that pass by during the cycle change-over period are assigned to the cycle during which the major portion of the platoon went by. This will underestimate the volume in one cycle and overestimate the volume in the next, but the characteristics of the platoon will be correctly represented.

Very large platoons (i.e., 20 or more) that span 2 cycles can be broken up into two parts, one in each cycle. The platoon assigned to the second cycle is then given the same lead vehicle as the original platoon. When the same type of vehicle is not present in the second part of the platoon, a dummy vehicle can be added to represent the correct lead-vehicle type, or a new leader assigned, depending on the purposes of the study.

Vehicle Type Categorization

When selecting the number of different types of vehicles that are to be distinguished and counted separately, one must trade-off between theoretical and practical requirements. From a theoretical standpoint one would like to divide the users of a highway into as many different types of vehicles as possible. Such a procedure allows for a highly detailed analysis of traffic behavior according to vehicle type. However, to set up a very large number of different types of categories is usually not very practical in terms of either data collection or analysis.

On the other hand, if one aggregates all vehicles into just one or two categories, one may sacrifice some important effects that result from having different types of vehicles use the same road.

In view of the difficulties that are associated with each extreme, one must make a compromise by selecting enough categories to distinguish the major differences, yet aggregate sufficiently so that the data-collection and analysis processes remain manageable.

Based on this argument it was decided in an Ontario study (2) to split up the different types of vehicles into four different groups: passenger cars (including vans and light 2-axle, 4-wheel trucks), larger trucks, recreational vehicles, and others. The choice of these four categories served our needs, yet the data-collection aspects were not unwieldy. The other category was included as a catch-all for undefined categories. Under special circumstances, more or fewer divisions can be made but in such cases a different data sheet from the one illustrated in Figure 2 would need to be constructed.

ADVANTAGES OF RADAR-PLATOON TECHNIQUE

The radar-platoon technique has both technical and economic advantages over existing state-of-the-art techniques. These advantages are briefly described below.

Technical Advantages

The radar-platoon technique creates an unbiased data set since the speed of each vehicle is represented. This is a significant improvement over most other techniques that only consider a sample of the vehicles.

The vehicle counts and the speed measurements are accurate as each vehicle is counted manually and the speeds are measured by using calibrated radar sets. No difference could be detected between measurements of independent observers. The recognition of platoon formations is a type of surrogate measure of service levels that could not be obtained from any of the previous techniques.

The simplicity of the technique eliminates the need for skilled operators, which in turn eliminates lengthy training requirements. Even unstable platoons can be handled with this method. The setup is highly mobile and, as a result, several different locations can be sampled without duplication of capital costs or difficult setups.

The thoroughness of the radar-platoon technique is illustrated by the sample output of summary data in Figure 5. The summary for each 5-min period is printed on a single line and consists of six basic sections. The first block describes the weather and surface conditions at the given date and time. Next, the total number of platoons and their breakdown according to lead-vehicle type are given. The third block has the total vehicle counts for each vehicle type and then for each direction. The fourth block gives the passenger car unit equivalents for each direction. The final two blocks of numbers represent the speed distribution for each 5-min period. This distribution is described by its average, low, and high speeds and then the percentile speeds from the 10th to the 90th.

Economic Advantages

The initial capital cost of the radar-platoon technique is small if no radar set needs to be purchased. The counter used for counting the opposing volume is inexpensive and the remainder of the equipment is available in the form of standard office supplies.

The variable cost of operating a monitoring station is also kept to a minimum because only one person is required to operate each such station. Each station also produces an additional count for opposing traffic. The data-collection technique yields a maximum return of data during low volumes when two-directional data are obtained.

CONCLUSIONS

The radar-platoon technique for collecting traffic data is fast, relatively inexpensive, requires a minimum of manpower, and provides a 100 percent sample of speeds. The distinction between different vehicle types allows for the evaluation of the effect of different vehicle type distributions on the behavior of traffic. The use of platoons as a measure of highway performance recognizes that individual vehicles may be affected by the other users of the highway. The count of the opposing traffic volume as well as the principal traffic flow provides a means of evaluating the impact of different directional traffic splits. The effect of slow moving vehicles can be determined through the analysis of the platoon lead-vehicle types.

The spot values of vehicle speed and traffic volume provide a direct means of establishing the relation between speed and volume for a set of specific highway conditions. The ability of mea-

Figure 5. Sample output of 5-min summaries and overall summary for data location.

**************			PLATOONS/LEADER					*****VEHICLE COUNTS******							*PCU PER HOUR*			***SPEED***			********PERCENTILE SPEEDS*********								
DATE	TIME	W/S	TOT	CA	TR	RE	OT	CAR	TR	RE	OT	MAI	OPP	TOT	MAI	OPP	TOT	AVG	LO	HI	10	20	30	40	50	60	70	80	90
905	945	O D	17	12	4	1	0	19	4	1	0	24	36	60	342.	513.	855.	91.	82.	104.	84.	86.	88.	89.	90.	91.	92.	95.	99.
905	950	O D	18	18	0	0	0	29	1	0	0	30	23	53	372.	285.	657.	88.	75.	110.	77.	79.	80.	82.	85.	91.	94.	95.	104.
905	955	O D	18	17	1	0	0	37	1	0	0	38	26	64	468.	320.	788.	83.	61.	98.	64.	76.	79.	81.	84.	87.	91.	93.	96.
905	1000	O D	17	16	1	0	0	28	3	0	1	32	47	79	432.	635.	1067.	83.	74.	95.	75.	76.	78.	80.	82.	84.	87.	89.	91.
905	1005	O D	17	14	3	0	0	27	5	0	0	32	42	74	444.	583.	1027.	87.	79.	108.	82.	83.	84.	85.	87.	88.	90.	91.	93.
905	1040	O D	18	13	5	0	0	30	6	0	0	36	28	64	504.	392.	896.	80.	70.	98.	71.	72.	75.	77.	78.	81.	84.	86.	94.
905	1045	O D	10	6	3	0	1	11	3	0	1	15	16	31	228.	243.	471.	81.	70.	91.	70.	73.	78.	82.	83.	83.	85.	86.	90.
905	1050	O D	14	12	1	1	0	24	2	1	0	27	30	57	354.	393.	747.	83.	69.	96.	71.	75.	76.	78.	80.	86.	91.	94.	95.
905	1055	O D	25	23	2	0	0	35	2	0	2	39	32	71	516.	423.	939.	90.	77.	112.	79.	83.	84.	85.	86.	89.	92.	98.	104.

***** SUMMARY OF LOCATION 7 E 1 *****

			VEHICLE		COUNTS-PERCENT		LEADS-PERCENT		VOLUMES		
5-MIN PERIODS	:	132	CARS	:	3994	90.8	1924	89.6	MAIN	:	4398
TOT PLATOONS	:	2148	TRUCKS	:	266	6.0	151	7.0	OPPOS	:	4023
			RECS	:	71	1.6	46	2.1	TOTAL	:	8421
			OTHER	:	67	1.5	27	1.3			

suring speed and obtaining detailed traffic counts for both directions during periods of low-traffic demand makes the radar-platoon technique a very efficient data-collection method. A 5-min counting cycle length was selected for the Ontario study because it is long enough for statistical aggregation while it remains short enough to reflect any short-term fluctuation.

REFERENCES

1. M. Van Aerde and S. Yagar. Comparing Manual and Automatic Techniques for Recording and Processing Traffic Data. Proc., Organization for Economic Cooperation and Development Symposium on the Effects of Speed Limits on Traffic Accidents and Transport Energy Use, Dublin, Ireland, Oct. 1981, 13 pp.
2. S. Yagar and M. Van Aerde. Field Studies on Two-Lane Highways: Volume/Capacity Relationships. Univ. of Waterloo, Waterloo, Ontario, Canada, Feb. 1981, 266 pp.

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Validation of Signalized Intersection Survey Method

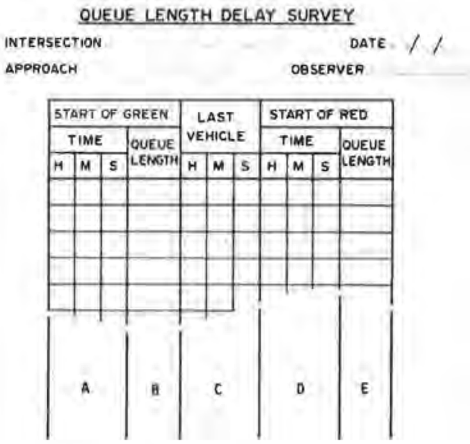
A.J. RICHARDSON AND N.R. GRAHAM

The development and validation of a manual survey method for the measurement of performance at signalized intersections are described. The method is easy to use in the field and simply requires that queue lengths and flows be measured at particular times within each cycle of the traffic signals on each approach being surveyed. The output of the program includes frequency distributions and summary statistics for measures of approach delay, stopped delay, stationary queue length, and various definitions of vehicular stops. The validation of the survey method was performed by comparison of survey measurements with measures obtained from a videotape recording of intersection operation. The comparison was performed in two stages. First, results obtained from the field method were compared with results obtained by viewing a videotape of the same traffic stream and extracting the survey data from the videotape. This comparison identified the field observer error in the survey method. Second, the results obtained by using the survey method with the videotape were compared with detailed path-trace information obtained from the same videotape. This comparison identified the theoretical error in the analysis calculations associated with the survey method. The comparison shows that the field survey method produced negligible observer error while the theoretical error in the survey method was quite small and well within the bounds dictated by practical traffic engineering requirements. It is concluded that the survey method is a simple, yet accurate, way of determining signalized intersection performance levels.

The evaluation of signalized intersection performance has long been an issue of concern, a concern that has intensified in recent years with the changing emphasis in urban transportation planning. Tightening budgetary constraints have led to reduced capital expenditure on transportation and have been partly responsible for the present emphasis on transportation system management. Moreover, there has been an increasing need to acquire more knowledge of demand so that the management of it is both publicly acceptable and consistent with an efficient allocation of resources, both privately and socially. To this end, energy conservation, environmental consequences, and equity (in terms of resource allocation, e.g., time savings) have become important concerns. The measurement of intersection performance, for example, should no longer be concerned solely with the motorist but with societal goals as a whole and with the equitable allocation of resources to individual members of society.

Determination of the level of performance of a signalized intersection has application in traffic engineering planning and design, in the study of the effects of physical and operational improvements,

Figure 1. Original field survey form.



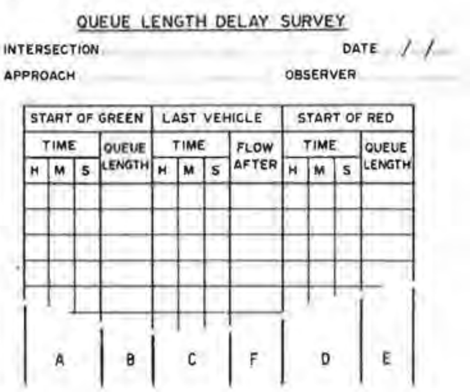
QUEUE LENGTH DELAY SURVEY

INTERSECTION DATE / /

APPROACH OBSERVER

START OF GREEN				LAST VEHICLE			START OF RED			
TIME			QUEUE LENGTH				TIME			QUEUE LENGTH
H	M	S		H	M	S	H	M	S	
A			B	C			D			E

Figure 2. Revised field survey form.



QUEUE LENGTH DELAY SURVEY

INTERSECTION DATE / /

APPROACH OBSERVER

START OF GREEN				LAST VEHICLE				START OF RED			
TIME			QUEUE LENGTH	TIME			FLOW AFTER	TIME			QUEUE LENGTH
H	M	S		H	M	S		H	M	S	
A			B	C			F	D			E

and in economic analyses. In particular, analyses of traffic-signal-control strategies, air and noise pollution related to vehicular traffic, road-user costs, evaluation of bus-priority schemes, effects of different objective functions for traffic-signal timing, and capacity and level-of-service calculations all require estimates of vehicular delay, stops, or other measures of performance.

The search for a measure of level of performance at an intersection takes its roots at the birth of the traffic engineering profession. An early work by Greenshields (1) used a 16-mm camera to capture traffic flow for subsequent analysis in the laboratory. Since then, a number of researchers (2-8) have continued the search for measures of performance. Reilly and others (8) have noted that in the evolution of performance-measurement techniques there have been two major problems. First, the definition of the performance-level criteria and second, the technique for obtaining such a measurement. Unfortunately, much of the previously reported work has not clearly defined either the phenomenon to be measured or the details of the measurement techniques.

To overcome many of the deficiencies of previous techniques, a new survey method was developed to assist in the evaluation of bus-priority signals (9). This method has previously been described by Richardson (10). Partly as a result of comments by Reilly (11), it was decided that this survey method should be subjected to a comprehensive validation study where survey method results would be compared with results obtained from a videotaped recording of intersection operation.

The purpose of this paper is to summarize the results of this validation study, which was sponsored by the Australian Road Research Board. This paper will only highlight the major points of the study; more complete details of the study may be found in a separate series of reports (12-17).

SURVEY METHOD DESCRIPTION

Before describing the validation study, it is necessary to provide details of the intersection survey method that is the subject of the validation study. The method has been initially described by Richardson (10). Since that time, however, the method has been substantially revised and improved, such that the present analysis program bears little resemblance to the one described earlier. A more up-to-date description of the theoretical background to the analysis procedure may be found in Richardson (14).

The survey method may be described in terms of three components: input, output, and special features of the analysis. The method offers two options with respect to the input data to be collected. The original version of the method, as described in Richardson (10), requires four items of data to be collected in any one cycle of the traffic signals on each intersection approach being surveyed. An example of the survey form used in the field is shown in Figure 1. At the start of the green phase, the time is recorded in column A and the number of vehicles stopped in the queue is recorded in column B. A mental note is made of the last vehicle in the queue at the start of the green and, when the queue moves off, the progress of this vehicle is noted. If this end-of-queue vehicle crosses the stop line before the signal changes back to red, then the time at which it crosses the stop line is recorded in column C. The time at which the signal changes back to red is then recorded in column D (column E, in this case, is left blank). If the end-of-queue vehicle does not cross the stop line before the lights change back to red, the time at which the lights change to red is recorded in column D and the number of vehicles in front of and including this vehicle when the new queue forms is recorded in column E (column C, in this case, is left blank). This process is repeated for every cycle in the survey period.

A limitation inherent in using the survey method with only these data is that it must be assumed that the arrival rate that was observed during the red period in each cycle continues through the following green period. Similarly, the move-off rate observed for vehicles up until the last vehicle in the queue at the start of the green is assumed to continue for all vehicles that depart in the current cycle. As noted by Reilly (11), both these assumptions may be invalid under certain circumstances (e.g., coordinated signals or flared intersection approaches). To enable the survey method to be used in such situations, a modification was made to the data-collection procedure that, while slightly increasing the workload in the field, allows for different arrival rates during the red and green phases and for changing move-off rates during the green phase. The modified field survey form is shown in Figure 2. The procedure is identical to that described above with respect to Figure 1 except that after the last vehicle in the queue at the start of the green has crossed the stop line, the observer counts the number of vehicles that then cross the stop line before the signals turn red and records this flow in column F. If the end-of-queue vehicles does not cross the stop line before the signals turn red, then columns C and F are left blank and column E is completed as before.

Given these relatively meager input requirements, the survey method produces a very comprehensive range of output statistics. Specifically, it produces frequency distributions and summary statistics (mean and standard deviation) for the following performance measures: approach delay, stopped delay, number of complete stops, number of effective stops (for use in fuel-consumption calculations), and maximum stationary queue length in the cycle. The total flow across the stop line and a complete record of signal phasing and timing are also obtained from the analysis program.

In calculating these output statistics, the analysis program (QDELAY) makes use of a number of special features that have hitherto not been incorporated in intersection survey method calculations. The analysis program is based on the construction of trajectory diagrams for vehicles that pass through the intersection. In calculating approach delay, use is made of the finding of Allsop (18) that, by considering vehicles with infinite acceleration and deceleration rates, the approach delay is equal to the length of the horizontal sections of the trajectories. The QDELAY program, however, extends this concept to include approach delay that is incurred after the vehicle crosses the stop line (i.e., while the vehicle is accelerating back up to cruise speed). This extension overcomes a problem that is evident in most, if not all, previous survey methods where approach delay is confined to being upstream of the stop line. Such a restriction may be most significant in situations where the average queue length is short, with a substantial amount of approach delay being incurred downstream of the stop line.

In calculating stopped delay, the time spent in deceleration and acceleration maneuvers is subtracted from the approach delay for each vehicle to reveal the time spent stopped. In many cases, the stopped delay may be zero when the vehicle does incur approach delay. The rates of acceleration and deceleration used in this calculation are user specified and may be chosen to suit the particular site in question.

The calculation of vehicular stops allows for two basic options. First, it is possible to calculate conventional measures of vehicular stops, such as the average number of complete stops per vehicle or the proportion of vehicles that are stopped. Second, because the number of vehicular stops is often used to calculate fuel consumption, it is possible to calculate a more appropriate measure of vehicular stops (termed effective stops) that allows for the effects of partial stops (that is, vehicles that slow down but do not completely stop) and queue-shuffling stops (where vehicles in saturated traffic conditions stop and "shuffle" forward several times before clearing the intersection). From the above discussion, it is obvious that the survey method can be used in saturated, as well as unsaturated, traffic conditions.

One final point that concerns the survey method is that, as described in Richardson (10), it can be used on approaches with more than one lane by defining a representative end-of-queue vehicle. The only restriction in using the method in this way is that flow characteristics (in particular, the move-off rate) should be similar in each lane.

VALIDATION OF STUDY DESIGN

The objectives of the validation study were to identify both the theoretical and observational errors in the survey method. To this end, the validation study incorporated three distinct data-collection phases:

1. Field observers used the survey method to measure intersection performance.
2. Concurrently, intersection operation was recorded on videotape. Later, observers viewed the videotapes in a laboratory and used the same survey method to record the level of intersection performance.
3. By using the same videotaped recording of intersection operation, independent measures of intersection performance were obtained, to a high level of precision, by tracing the individual movements of a sample of vehicles through the intersection.

By comparing the results of phases 1 and 2, the observational error could be ascertained. A comparison with the results from phases 2 and 3 would reveal the theoretical error in the survey method calculations.

The main difficulty in the survey design was to find study site locations, given a rather formidable list of constraints in camera location, traffic flow conditions, general site characteristics, and time and budget constraints. Two isolated, signalized intersection sites were finally chosen (Nicholson Street and Beaconsfield Parade). Both sites are located approximately 3 km from the Melbourne central business district (CBD). The sites were chosen to give a wide variety of traffic conditions, including both peak and off-peak periods. At the Nicholson Street site, a total of 15 h of data was collected, comprising 4 h in the morning peak, 4 h in the evening peak, and 7 h during the afternoon off-peak. At the Beaconsfield Parade site, a total of 4 h of data was collected, all in the morning peak period. At the Nicholson Street site, where there were two approach lanes, data were collected separately for each lane. At the Beaconsfield Parade site, where there were three through lanes, data were collected for both separate and multiple-lane situations. By allowing for different combinations of the times of the surveys and the lane configuration, a total of 50 data sets was obtained for comparison.

The collection of data by using the survey method in the field was relatively straightforward by using the techniques described earlier in this paper [and in Richardson (15)]. When using the survey method in the laboratory, observers made full use of the stop-frame action of the videotape playback in order to make observations with great accuracy (e.g., to count the number of vehicles in a long queue).

In using the path-trace method, a number of factors needed to be accounted for. The definition of an approach-delay section was an essential prerequisite to the collection of path-trace data. The upstream end of the section was defined to be an easily identified point some 100-m upstream from the longest expected queue. The downstream end was defined to be the stop line at the intersection. This was necessary in order to ensure that the video-camera was close enough to the intersection to obtain a reasonable view of the stationary queue. The definition of the approach-delay section in this way, however, required that the ability of the QDELAY program to calculate approach delay incurred after the stop line be neglected for the validation study comparisons.

In using the path-trace method, it is necessary to make an assumption about the free speed of vehicles through the approach-delay section in order to calculate the delay in this section. The free speed was calculated in two ways. First, the speeds of vehicles that passed through the section unimpeded were obtained from the path-trace records, and the free speed was set equal to the 85th percentile point of the distribution of unimpeded speeds for that approach and time of day. Second, as part of a study to determine deceleration rates at the intersections, an estimate was obtained of the speed at which all vehicles, whether impeded or not, approached the intersection. Both methods gave very similar results at both Nicholson Street and Beaconsfield Parade.

Extraneous vehicles (that is, vehicles that do not cross both the upstream and downstream ends of the approach-delay section) were eliminated from all path-trace calculations because of the difficulties of defining delays for such vehicles. At each site, extraneous vehicle activity was approximately 20 percent of the total flow across the stop line. Since many delay measurements were made on a lane-by-lane basis, it was necessary to allocate each vehicle in the path-trace survey to a particular lane. As with Reilly and others (8), vehicles were allocated to the lane in which they crossed the stop line. Although it is acknowledged that the path-trace method does not give completely accurate results, it was considered that such results were as close to the true situation as could be obtained within reasonable budgetary limits. Support for this contention may also be found in Reilly and others (8), which indicates that path-trace results provide a reasonable basis against which to compare the survey method results.

Figure 3. Comparison of survey results when used in field and in laboratory.

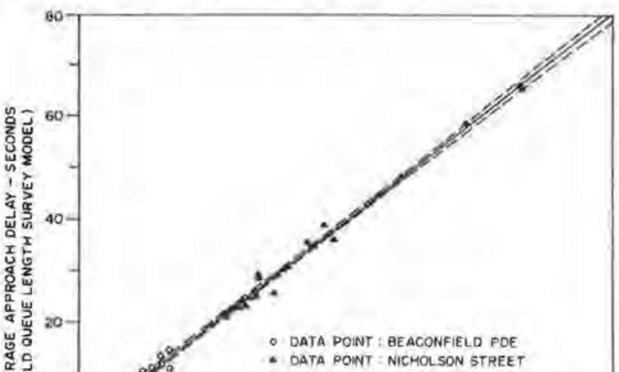


Table 1. Regression of path-trace results against measures obtained from use of survey method in laboratory.

Variable	Regression Coefficients[a]		Correlation Coefficients		
	Intercept a	Slope b	r^b	r^2	E
Approach delays					
Total	-456 ± 847	0.94 ± 0.06	0.98	0.96	0.92
Avg	-1.72 ± 1.44	0.99 ± 0.05	0.99	0.97	0.95
SD	-0.45 ± 1.23	0.91 ± 0.06	0.97	0.95	0.87
Stopped delays					
Total	43.3 ± 4563	1.06 ± 0.05	0.99	0.98	0.97
Avg	0.50 ± 1.14	1.01 ± 0.05	0.98	0.97	0.96
SD	-0.80 ± 1.28	1.01 ± 0.07	0.97	0.94	0.93
Number of stops					
Total	9.00 ± 18.5	1.06 ± 0.04	0.99	0.98	0.96
Avg	0.10 ± 0.04	0.92 ± 0.05	0.98	0.96	0.91
SD	0.18 ± 0.10	0.56 ± 0.20	0.62	0.39	-0.63
Number stopped					
Total	-0.80 ± 19.0	1.11 ± 0.05	0.99	0.98	0.94
Proportion	0.13 ± 0.04	0.88 ± 0.06	0.97	0.94	0.83
Vehicular volume–total	0.00 ± 0.00	1.00 ± 0.00	1.00	1.00	1.00

[a]Coefficient ± 95 percent confidence interval for coefficient.
[b]r always significantly different from zero at 0.05 level (Fisher's Z-test).

VALIDATION OF STUDY RESULTS

In presenting the results of the validation study, two different types of analyses are described. The first is a comparison of the performance measure summary statistics obtained for each of the 50 data sets (i.e., the 50 combinations of survey site, survey time, and lane configuration). The second analysis is a comparison of the frequency distribution predicted and observed for each of the performance measures.

In presenting these results, only those obtained by using the expanded survey method are shown (i.e., by using the survey form shown in Figure 2). For the study sites, there was no appreciable difference between results obtained by using either of the survey forms, mainly because each intersection was isolated from upstream intersections and hence the arrival rates in the red and green periods were approximately equal. It should also be noted that there was no difference between results obtained by using the survey method in the field and in the laboratory (see, for example, Figure 3). This implies that there was little or no observer error in recording queue lengths or signal timings in the field. Independent comparison of queue-length estimates in the field and in the laboratory confirmed this impression, although queue-length estimates in the field were marginally smaller than those in the laboratory. Signal-timing observations in the field were also quite accurate. It should be noted, however, that digital stopwatches were used in the field surveys and this eliminated many timing errors that might have occurred if normal wrist watches had been used for timing. Also, the data-entry program used in QDELAY automatically detects obvious timing errors and allows for correction of these errors. A more complete description of the comparison between survey method results obtained in the field and in the laboratory may be found elsewhere (16).

To examine the theoretical error in the survey method calculations, a comparison of summary statistics is presented both in tabular and graphical fashion. Table 1 summarizes the results of regression analyses conducted when measures of delay, stops, and vehicular volume obtained from the survey method in the laboratory were compared with the same measures obtained from the path-trace method. These regression analyses were conducted with all 50 data sets (i.e., both sites, all times of day, and all lane configurations), each contributing one data point to the analysis.

The regression equation used was of the form

$$Y = a + bX \tag{1}$$

where

Y = performance measure obtained from the survey method in the laboratory,
X = performance measure obtained from the path-trace method, and
a,b = estimated regression coefficients.

As can be seen in Table 1, most measures were predicted by the survey method with a high degree of consistency, as indicated by the high values of r^2. More importantly, the survey method and the path-trace method give nearly equal values of the performance measures, as indicated by the high value of E. This value, termed the coefficient of efficiency (19), may be used to test for bias in the regression relation. If the results from the survey method and the path-trace method are highly correlated but biased (i.e., the data points do not lie evenly around the Y = X line), then E will be much less than r^2. If there is no bias in the relation, then $E = r^2$. With the exception of the regression for the standard deviation of the number of stops per vehicle, the high values of r^2 and E indicate excellent agreement between the survey method and path-trace results.

Two other indications of the agreement between the two survey methods may be seen in the size of the intercepts and slopes of the regression lines. If there was perfect agreement between the two methods, the intercept would be equal to zero and the slope would be equal to one. It can be seen that, in most cases, the 95 percent confidence limits include the desired value of either the intercept or the slope, which indicates excellent agreement.

The conclusions that may be drawn from Table 1 may be reinforced by reference to Figures 4 through 7, which show the data points, regression lines, and confidence limits for a number of different performance measures. For each of the measures shown, which are the most important outputs of the survey method, it is obvious that there is quite good agreement between the two methods of collecting data on intersection performance.

The second type of analysis of the results is to compare the frequency distributions of the performance measures obtained from each of the survey methods. In comparing these distributions for ap-

Figure 4. Comparison of average approach delays.

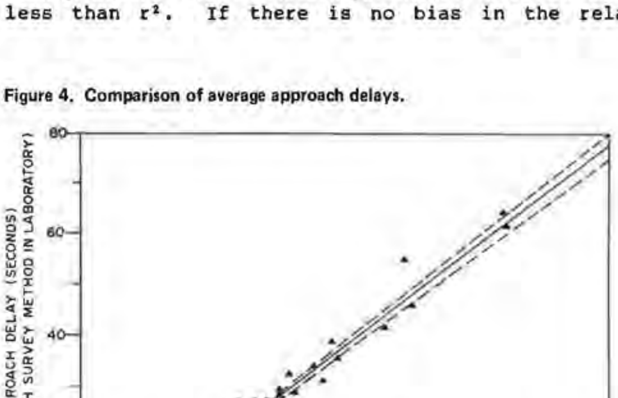


Figure 5. Comparison of standard deviations of approach delay.

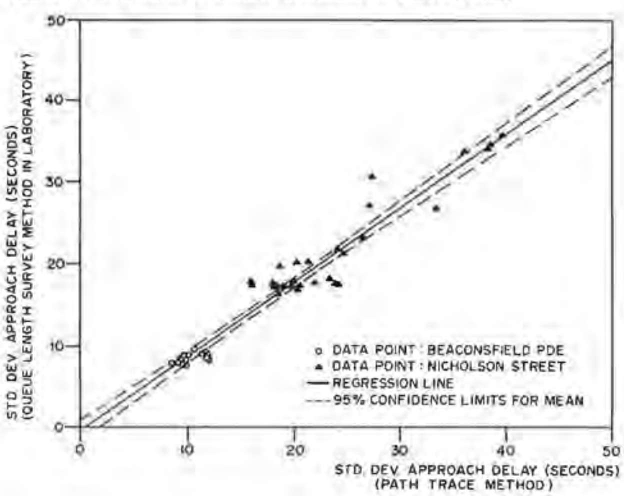


Figure 6. Comparison of average stopped delays.

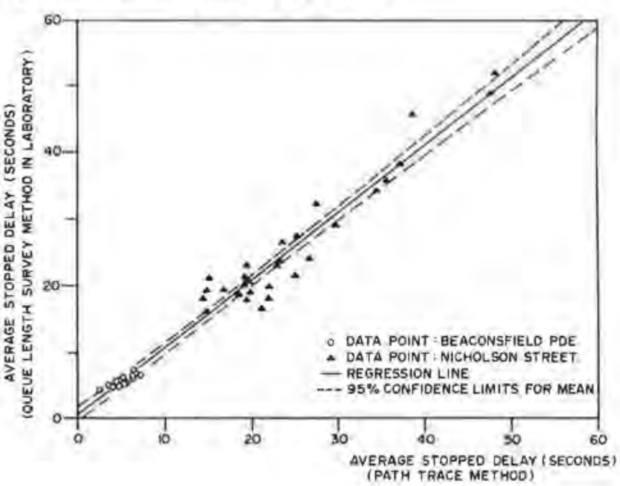


Figure 7. Comparison of number of complete stops per vehicle.

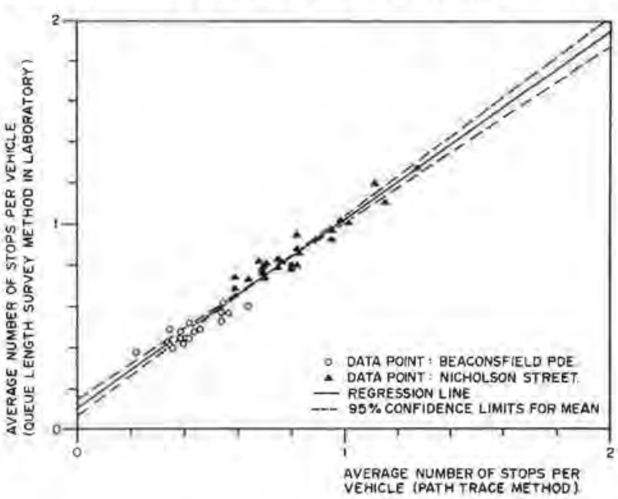


Figure 8. Comparison of approach-delay distributions (Nicholson Street).

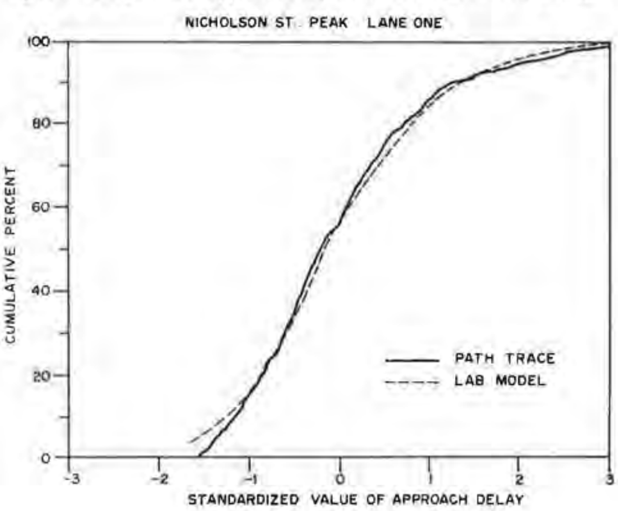


Figure 9. Comparison of approach-delay distributions (Beaconsfield Parade).

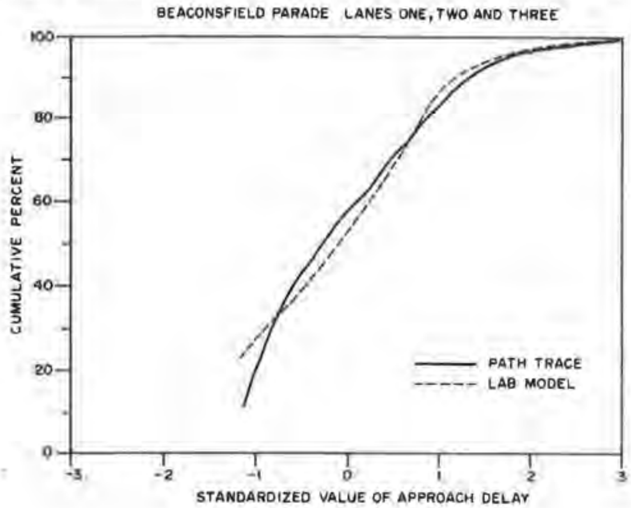


Figure 10. Comparison of stopped-delay distributions (Nicholson Street).

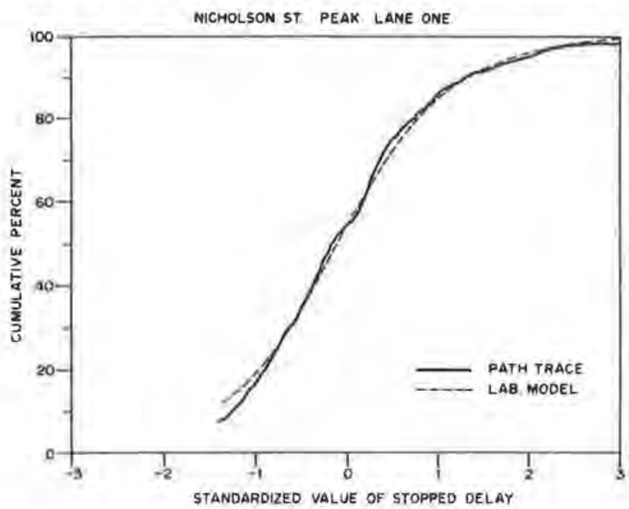
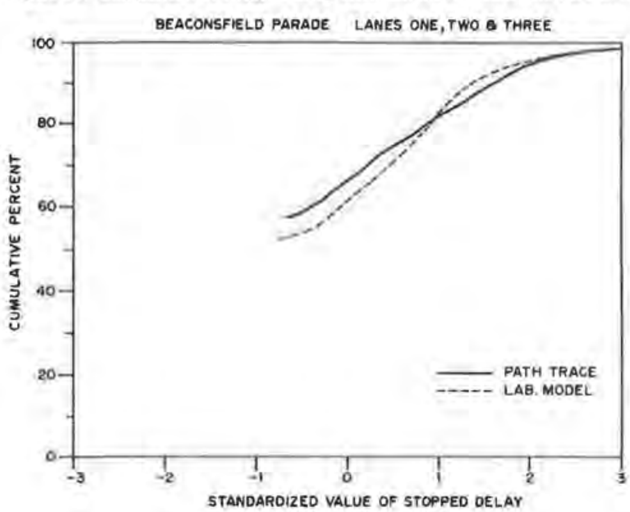


Figure 11. Comparison of stopped-delay distributions (Beaconsfield Parade).



proach delay and stopped delay, the analysis was conducted separately for each site, time of day, and lane across stop line. Although it is not possible to present all these results [see Richardson and Graham (16)], Figure 8 and 9 show typical results for approach delay obtained for Nicholson Street and Beaconsfield Parade, respectively. It is clear that the agreement between the shape of the distributions is better at the Nicholson Street site, although it is far from poor at Beaconsfield Parade. Note that the results shown for Beaconsfield Parade are for the case where all three lanes are combined in the one data set, thus necessitating the use of a representative end-of-queue vehicle, as described earlier. At both sites, the approach-delay distribution shows a characteristic skew to the right.

The distributions of stopped delay for typical cases at both sites are shown in Figures 10 and 11. Again it can be seen that Nicholson Street data produce better agreement than Beaconsfield Parade data, when the proportion of vehicles that suffer no stopping delay is slightly overpredicted by the queue-length survey method. However, considering that the distributions from the queue-length survey method are synthesized from the relatively simple input data whereas the path-trace distributions are constructed from measures of individual vehicle performance, the agreement between the distributions is quite satisfactory.

CONCLUSION

This paper has described the validation of a survey method for the measurement of performance at signalized intersections. The input and output of the method have been described and some features of the analysis program have been discussed. The conduct of the validation study has been described and some of the results of the study are presented. On the basis of the results presented (and those contained in other, more complete, reports), it is concluded that the survey method produces a wide array of output statistics to a high degree of accuracy (when compared to observations by using a path-trace method). Despite the comprehensive nature of the outputs, the input to the survey method is relatively simple and requires few resources in terms of personnel and equipment. It is anticipated that the survey method should find ready application in many signalized intersection survey studies.

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REFERENCES

1. B.D. Greenshields. A Photographic Method of Investigating Traffic Delays. Proc., Michigan Highway Conference, 1934.
2. D.S. Berry and C.J. van Til. Comparison of Three Methods for Measuring Delay at Intersections. Traffic Engineering, Vol. 25, 1954, pp. 93-99.
3. D.S. Berry. Field Measurement of Delay at Signalized Intersections. Proc., HRB, Vol. 35, 1956, pp. 505-522.
4. G.S. Sagi and L.R. Campbell. Vehicle Delay at Signalized Intersection: Theory and Practice. Traffic Engineering, 1969, pp. 32-40.
5. H. Sofokidis, D.L. Tilles, and D.R. Gerger. Evaluation of Intersection-Delay Measurement Techniques. HRB, Highway Research Record 453, 1973, pp. 28-48.

6. H.D. Robertson and W.G. Berger. A Manual Technique for Measuring Intersection Delay. Traffic Engineering and Control, Vol. 17, Oct. 1976, pp. 396-398.
7. M.G. Buehler, T.J. Hicks, and D.S. Berry. Measuring Delay by Sampling Queue Backup. TRB, Transportation Research Record 615, 1976, pp. 30-36.
8. W.R. Reilly, C.C. Gardner, and D.H. Kell. A Technique for Measurement of Delay at Intersections. FHWA, Final Rept., 1976. NTIS: PB-265 701.
9. A.J. Richardson and K.W. Ogden. An Evaluation of Active Bus-Priority Signals. TRB, Transportation Research Record 718, 1979, pp. 5-12.
10. A.J. Richardson. Measurement of the Performance of Signalized Intersections. TRB, Transportation Research Record 699, 1979, pp. 48-58.
11. W.R. Reilly. Comments on Measurement of the Performance of Signalized Intersections by A.J. Richardson. TRB, Transportation Research Record 699, 1979, p. 59.
12. N.R. Graham and A.J. Richardson. A Review of Measures of Signalized Intersection Performance. Monash Univ., Clayton, Victoria, Australia, Civil Engineering Working Paper 80/5, 1980.
13. N.R. Graham and A.J. Richardson. Signalized Intersection Survey Method: Validation Survey Design and Administration. Monash Univ., Clayton, Victoria, Australia, Civil Engineering Working Paper 80/15, 1980.
14. A.J. Richardson. Signalized Intersection Survey Method: Theoretical Background. Monash Univ., Clayton, Victoria, Australia, Civil Engineering Working Paper 80/13, 1980.
15. A.J. Richardson. Signalized Intersection Survey Method: Field Survey Users Manual. Monash Univ., Clayton, Victoria, Australia, Civil Engineering Working Paper 80/14, 1980.
16. A.J. Richardson and N.R. Graham. Signalized Intersection Survey Method: Empirical Results of Validation Study. Monash Univ., Clayton, Victoria, Australia, Civil Engineering Working Paper 80/16, 1980.
17. A.J. Richardson and N.R. Graham. Objectives and Performance Measures in Urban Transport Systems. Seminar on Fuel Consumption, Vehicle Emissions, Traffic Management, and Vehicle Design; Society of Automotive Engineers, Australia, and the Australian Road Research Board, 1980.
18. R.E. Allsop. Delay at a Fixed Time Traffic Signal--Part I: Theoretical Analysis. Transportation Science, Vol. 6, No. 3, 1972, pp. 260-285.
19. A.P. Aitken. Assessing Systematic Errors in Rainfall-Runoff Models. Journal of Hydrology, Vol. 10, 1973, pp. 121-136.

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Computer-Controlled Videotape Display: An Innovation in Traffic Analysis

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Although videotape equipment has been available to traffic researchers and engineers for over a decade, its uses have been limited to routine applications. However, the recent development of microcomputers and interface equipment facilitate the use of videotape (and videodisc) in research applications. Current research under contract to the Iowa Department of Transportation is detailed where computer-videotape simulations of uncontrolled intersections elicit responses by a sample drawn from a public location. Data are presented to demonstrate (a) the efficacy of the videotape-computer research approach as well as (b) useful findings that suggest the presence of word-oriented versus symbol-oriented subgroups in the adult population, each having very different responses to various warning signs.

Television and videotape have been used as traffic engineering data-collection tools in a variety of ways within the past decade as portable camera-recorder systems became generally available (1-5). Some of these uses have included collecting data on the speed of vehicles; lane placement of vehicles; license-plate vehicle identification for monitoring vehicles through a portion of a system; accident surveillance on bridges, tunnels, and freeways; and emergency traffic operations coordination. Videotape is being commonly used in education and training activities. This use is not, however, as extensive as is commonly thought by persons outside of education. In this paper, we presume such use to be common knowledge. In a similar fashion, the general availability of small personal computers (32K-64K memory) for use in both traffic engineering and education activities is assumed to be common knowledge. What is new on the technological scene is an interface board to permit a microcomputer to control a new generation of video player-recorders. This combination provides a new analysis tool (6). This paper outlines how this new tool has been incorporated into an innovative analysis of rural road signing through some creative computer programming.

PROBLEMS IN SIGNING

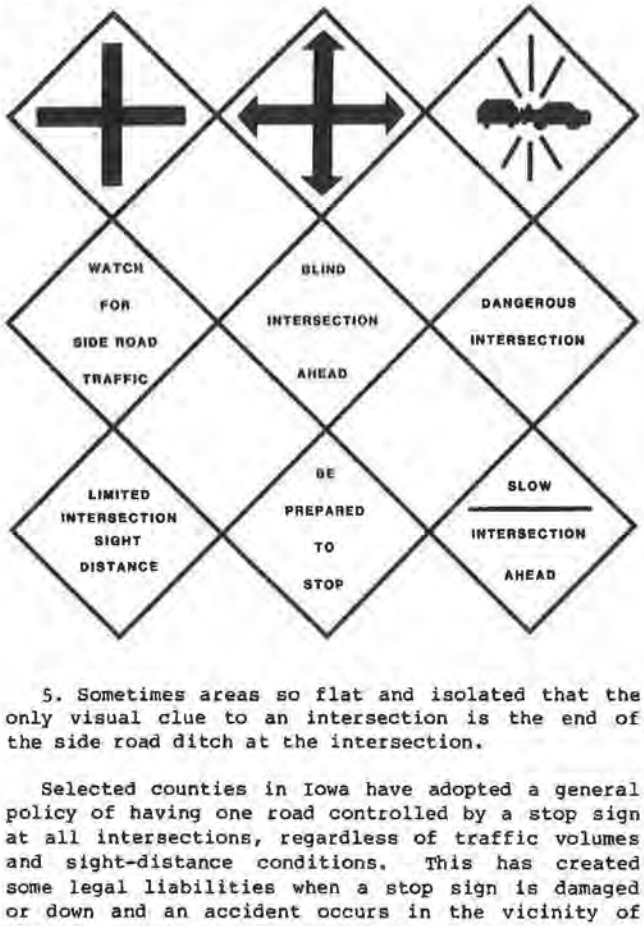
Several Iowa counties were frustrated in their attempts to communicate with people driving their extensive network of low-volume gravel rural roads. When these low-volume gravel roads intersect in the rolling Iowa terrain, a variety of factors interact to create seasonal (or sometimes continuously) hidden intersections. Some examples include the following:

1. Tall corn growing, planted to the very edge of the right-of-way (or perhaps in the right-of-way);
2. Trees at farmsteads in the corner quadrants of the intersecting roads;
3. Sharp curves within narrow cuts;
4. Densely wooded areas on curves; and

Table 1. General intersection site approach characteristics.

Site	Iowa County	Characteristic
1	Story	Cropland and pasture; intersection located at bottom of sharp sag vertical curve in all four directions
2	Boone	Cropland; intersection located at top of crest vertical curve in all four directions
3	Boone	Cropland and timber along stream; intersection is beyond short crest adjacent to a timber area
4	Boone	Timber, sharp curvature to road with steep grade; very hilly area along the Des Moines River; intersection is on curve in dense woods at end of sharp downgrade
5	Crawford	Grassland and cropland in sharp to rolling Missouri River region hills; intersection is at end of curving downgrade through cut in hill with crest location with respect to crossroad
6	Benton	Cropland in flat terrain; intersection is obscured by tall corn growing in fields adjacent to intersection approach

Figure 1. Array of signs in statistical design.



5. Sometimes areas so flat and isolated that the only visual clue to an intersection is the end of the side road ditch at the intersection.

Selected counties in Iowa have adopted a general policy of having one road controlled by a stop sign at all intersections, regardless of traffic volumes and sight-distance conditions. This has created some legal liabilities when a stop sign is damaged or down and an accident occurs in the vicinity of the sign. It is also difficult to justify stop signs that are not warranted by the Uniform Manual on Traffic Control Devices (MUTCD) (7) when litigation arises in court over accidents in the intersection.

One Iowa county has erected a warning sign with the legend Dangerous Intersection at a large number of low-volume rural roads in an attempt to communicate a need for increased attention to drivers who approach the intersection. Such a sign can very easily become a liability rather than an asset if a lawsuit erupts over an accident at an intersection that has this sign.

One county has experimented with using the standard crossroad sign (7, Section 2C-14) at low-volume rural road intersections for which drivers complain about near misses or potential collisions. Generally, these intersections do not warrant stop signs or are they amenable to expensive changes. Such use of the standard crossroad sign is also a potential liability problem since this sign is intended to be used on a through highway intersection approach, and it implies that drivers on one of the intersecting roadways will be required to stop.

In an effort to identify what the signing needs were at local uncontrolled road intersections and to establish some measure of what is effective sign communication, the Iowa Department of Transportation issued a contract to Iowa State University to investigate signing for these previously identified conditions. In order to develop a research design that would permit examination of these problem areas, videotape was combined with an interactive computer display to simulate approaching an intersection as a variety of signs are shown and respondent evaluation criteria are provided.

EXPERIMENTAL DESIGN FACTORS

A primary factor was the variation in terrain that produces regional variations in the topography that surrounds an obscured low-volume rural road intersection in Iowa. The original experimental design intent was to tape intersection approaches in the central part of Iowa (which tends to be flat to rolling cropland), in the northeastern part of Iowa (which tends to have sharp hills with dense woods), in the western part of the state (which has sharp hills with grasslands), and in the southeastern part of Iowa (characterized by short, sharp rises and falls in terrain in a strip-mining region). Careful selection of intersection approaches permitted use of four intersections within a 20-mile radius to represent four very different terrain conditions. Table 1 lists the general characteristics of each site.

A second factor involved our attempt to distinguish between the communication needs of the local driver (who would be familiar with the intersection) as opposed to the driver who might never have seen the intersection before. It was possible to simulate these two conditions by videotaping a fixed establishing shot of each intersection as well as a long-drive approach to the warning-sign location. The establishing shot provided the sample respondent with a close-up view of the intersection followed by a distant view of a vehicle traversing the intersection and turning. Thus, the respondent knew the intersection. The long-drive approach consisted of a 10- to 15-s tape segment filmed through the windshield and over the driver's right shoulder to show the view of the road that approaches the hidden intersection. When the warning-sign location is reached the camera pans across to the sign and zooms in to fill the field of view with a blank sign face. A respondent who represents the familiar driver sees both the establishing shot and the long-drive approach before responding to his or her first sign. A respondent who represents only unfamiliar drivers sees the long-drive approach only and begins sign responses.

Another control factor consisted of either the initial display of a symbol or word-legend sign. One-half of the sample is programmed to see a tape segment first ending with the crossroad sign and the other half first sees a Dangerous Intersection sign. The complete array of signs presented is shown in Figure 1. These two signs were selected for the initial variations of word versus symbol because they are currently being used to a limited degree to warn drivers who approach low-volume intersections that have potential traffic problems. The selection of the remaining signs was based on a pretest among a sample of engineers and social scientists that considered both a hypothetical and an actual variety of legends already in use.

After each sign is shown on the television screen the computer halts the tape player in the pause mode, returns control of the television screen to the computer, and presents a display on the screen that asks for an evaluation of the sign presented. The respondent is asked to evaluate the sign as

1. Very good,
2. Good,
3. Cannot decide or no opinion,
4. Bad, or
5. Very bad.

The computer then moves the tape back into the play mode in order to show a very short (3-5 s) portion of the driving approach to the hidden intersection followed by another sign. This process continues until each respondent has examined all nine signs for one intersection site.

After the respondent has completed the rating of each of the nine signs, the computer flashes a question that asks the person to select the best sign from among those shown, which are as follows:

1. Plus,
2. Arrows,
3. Crashing cars,
4. Watch for Side Road Traffic,
5. Blind Intersection Ahead,
6. Dangerous Intersection Ahead,
7. Limited Intersection Sight Distance,
8. Be Prepared to Stop, or
9. Slow - Intersection Ahead.

The computer informs the subject that an attendant has a set of signs should they wish to refresh their memory. After the respondent has selected the best sign, the computer asks each person to select the worst sign. These ratings permit weighting of the individual evaluation given each sign as it is presented.

At this point, the computer asks the person to decide whether, in their estimation, the intersection actually should have a sign to be safe. The possible responses offered are

1. Definitely needs a sign;
2. Probably needs a sign;
3. Don't know, no opinion, don't care;
4. Probably does not need a sign; and
5. Definitely does not need a sign.

Since one-half of the respondents see the establishing shot at the beginning and one-half at the end of each intersection site sequence, this question, which relates to the need for the sign, will permit weighting the sign responses according to perceived need. In a limited pretest exercise, one person responded that a particular sign was good and that it was the best of the nine signs but that in his opinion the intersection did not need a sign at all. Preference studies must have a mechanism by which preferences can be related to some normalizing scale to estimate salience of the performance.

STATISTICAL ASPECTS OF PRESENTATION

Twenty-four separate displays were created through the combination of computer control of the videotape player and videotape editing. Each site tape (there were six) was edited to have an establishing shot, then a long-drive approach to the intersection, then a symbol-sign presentation (crossroad sign), then a word-sign presentation (Dangerous Intersection sign). Then a fixed sequence of other signs is presented (each site has a different sequence of the same signs), then the crossroad sign, then the Dangerous Intersection sign, then the establishing shot. The computer is programmed to display to the sample respondents the following:

1. Respondent 1: Establishing shot, long drive up, crossroad sign, site 1 sequence for seven signs, Dangerous Intersection.
2. Respondents 2-6: Same as 1 except that each site (2-6) has a different sequence to the seven signs.
3. Respondents 7-12: Establishing shot, long drive up, Dangerous Intersection sign, site sequence for seven signs, crossroad sign.
4. Respondents 13-18: Long drive up, crossroad sign, site sequence for seven signs, Dangerous Intersection sign, establishing shot.
5. Respondents 19-24: Long drive up, Dangerous Intersection sign, site sequence for seven signs, Dangerous Intersection sign, establishing shot.

The pattern began again with respondent 25. This permitted control of a wide variety of factors.

The original research intent was to participate in a number of local county festivals and fairs across the State of Iowa. However, an opportunity was created for our participation in the Iowa Department of Transportation display at the 1981 Iowa State Fair in Des Moines. Respondents were obtained from adult drivers between the ages of 14 and 99 who visited the fair display.

A total sample of 405 persons was obtained. Seventy percent of the respondents were male, 24 percent were female, and six percent of the responses were made up of combined response from both a male and a female. The crossroad sign (symbol) was seen first on the tape display by 222 respondents while 183 saw the Dangerous Intersection (word) sign first. Each site represented from 15.8 to 17.5 percent of the responses. Sixty-six of the 99 Iowa counties were represented in the sample.

RESULTS

Table 2 illustrates that the sample respondents exhibited significant preferences for some signs over others. Since the crossroad sign and the Dangerous Intersection sign were given more positive ratings, the possibility existed that seeing a sign first or last implied to a respondent a researcher preference. When the mean scores for each sign were analyzed, controlling for whether a word or a symbol sign was seen first and whether or not the establishing shot was seen first, no statistically significant differences were to be found among the mean rating scores.

Table 3 contains the results of the forced choice as to the best and the worst sign for the intersection as seen on tape. The grouping of the data around conflicting preferences leads to the hypothesis that the respondent sample contained both word and symbol persons. A pattern of choice responses was created by using best-on-symbol/worst-on-word ratings (and vice versa) with corresponding highly favorable rating of the crossroad sign (and inversely a highly unfavorable rating of the Dangerous Intersection sign) to select consistent preference persons. The selection process resulted in 40 word persons and 49 symbol persons being identified.

Table 2. Mean ratings for signs by site used.

Site	No. of Re-spon-dents	Sign Shown: Crossroad		Watch for Side Road Traffic		Blind Inter-section Ahead[a]		Limited Inter-section Sight Distance[b]		Be Prepared to Stop[c]		Slow Inter-section Ahead[d]		Arrows[e]		Crashing Cars[f]		Dangerous Intersection[g]	
		Mean	Rank	Mean	Rank	Mean	Rank	Mean	Rank	Mean	Rank	Mean	Rank	Mean	Rank	Mean	Rank	Mean	Rank
1	73	2.23	1	2.95	6	2.38	2	3.37	8	2.86	5	2.41	3	3.17	7	3.87	9	2.54	4
2	67	2.26	1.5	3.37	8	2.50	4	3.40	8	2.62	5	2.37	3	3.11	7	2.26	1.5	2.79	6
3	71	2.60	2	3.01	7	2.63	3.5	2.90	5	3.59	9	2.63	3.5	2.94	6	3.40	8	2.12	1
4	64	2.46	3	3.37	8	2.82	5	2.37	1	3.56	9	2.95	6	2.57	4	3.32	7	2.40	2
5	66	2.19	1	3.00	6	2.36	3	3.72	9	2.95	5	2.31	2	3.37	8	3.01	7	2.57	4
6	64	2.20	1	3.18	6.5	3.46	8	2.53	4	2.81	5	3.67	9	2.23	2	3.18	6.5	2.29	3
Rank			1		9		3		5.5		7		4		5.5		8		2

Note: For mean ratings, 5 = most disliked, 1 = most liked.

[a]Statistical significance: $F = 7.53 < 0.001$. [b]$F = 17.47 < 0.001$. [c]$F = 6.78 < 0.001$. [d]$F = 14.48 < 0.001$. [e]$F = 7.45 < 0.001$. [f]$F = 12.78 < 0.001$. [g]$F = 3.07 < 0.001$.

Table 3. Respondent rating of best and worst signs.

Sign	Rating (%) Best	Worst
Crossroad (symbol)	26.4	6.2
Arrows (symbol)	10.9	11.4
Crashing cars (symbol)	8.4	34.3
Watch for Side Road Traffic	3.0	2.5
Blind Intersection	17.0	1.2
Dangerous Intersection	19.3	1.0
Limited Intersection Sight Distance	1.2	36.5
Be Prepared to Stop	7.7	4.0
Slow – Intersection Ahead	6.2	3.0

Given that both groups represent about 10 percent of the sample, it was hypothesized that they represent the tails of a normal distribution of communication modal preference. Traffic engineers now have available a method to determine the degree of communication between driver and signs without actually endangering lives or generating legal hazards through field testing.

CONCLUSIONS

The research reported in this paper demonstrates that this method is effective in providing driver evaluation of proposed traffic-control changes. Since the concept has been under development for over five years, it is gratifying to conclude that the method does indeed work.

Data analysis results of this research indicate the presence of word-oriented and symbol-oriented persons among the driving population. It is concluded that further analysis of this behavioral and perceptual character of drivers will be a future critical issue in making standard traffic signing and signal displays more effective.

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REFERENCES

1. S.L. Ring. Video System's Application to Traffic Engineering. Presented at the American Society of Civil Engineers Convention, Kansas City, MO, paper 2393, Oct. 21-25, 1974.
2. R.E. Danielson, R.S. Brown, R.M. Hanchett, and W.C. Allinson. A Closed-Circuit Television System for Traffic Surveys. Traffic Quarterly, Vol. 28, Jan. 1974, pp. 87-100.
3. W.T. Baker, J.C. Williams, and C.W. Dale. Time-Lapse Television: A Highway Engineering Tool. Public Roads, Vol. 37, Dec. 1972, pp. 119-121.
4. K.S. Opeila, S. Khasnabis, and T.K. Datta. Determination of the Characteristics of Bicycle Traffic at Urban Intersections. TRB, Transportation Research Record 743, 1980, pp. 30-37.
5. K.A. Brewer and S.L. Ring. Erratic Maneuvers as an Interchange Design Feedback Tool. TRB, Transportation Research Record 562, 1976, pp. 49-58.
6. K.A. Brewer and W.F. Woodman. The Future Role of Interactive Simulation as a Device to Improve Household Quality of Life Decisions: Where Dynamic Forecasting is Going. In The Quality of Life: Systems Approaches of Applied Systems and Cybernetics--Volume 1 (Proc., International Congress of Applied System Research and Cybernetics), Dec. 1981, pp. 482-486.
7. Uniform Manual on Traffic Control Devices. FHWA, 1978.

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