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## PART 1 <br> Driver Performance Studies

## Foreword

This Record is divided into three parts, each covering a distinct research area: driver performance, pedestrians, and motorist information systems.

Several aspects of driver performance and behavior are discussed in the papers of Part 1. Differences between closed-course driving and various laboratory tests of 30- to 51- and 74to 83-year-old drivers were identified by Ranney and Pulling. Hunter-Zaworski, who deals with similar age groups and with subjects who have a restricted range of neck movement, examined driving performance by measuring decision time at simulated T-intersections.

A model for use in designing freeway exit ramps is described by Fazio et al. Driver behavior parameters were an integral part of the model, along with roadway, traffic, and vehicle characteristics.

Zegeer et al. determined the ability of trucks of various configurations to negotiate rural roads with restrictive geometry. By following selected trucks through 60 sites, the authors could analyze the effects of larger trucks on rural traffic operations and safety.
The remaining papers in Part 1 deal with speed and traffic volume and their impact on crashes. On the basis of data collected at 50 urban and rural sites (with $25-$ to $55-\mathrm{mph}$ speed limits), Harkey et al. determined travel speed characteristics, compliance with posted speed limits, and the point of minimum accident risk. Using these data, they assessed current speed zoning criteria.

Freedman and Esterlitz evaluated the effect of increased Interstate speed limits in three states. Nondetectable radar was used to measure free-flowing vehicle speeds during daytime off-peak periods.

Hall and Pendleton determined the nature of the relationship between hourly traffic volumes and hourly accident rates on rural highways in New Mexico.

Using time series analysis, McKnight and Klein compared fatal accidents, injury accidents, vehicle miles traveled, and vehicle speeds for the preceding 5 years with similar data for 1 year following the increase in the national maximum speed limit. In the last paper of Part 1 , Sidhu uses linear regression to compare accidents from the prior 5 years with similar data from the year following the change to 65 mph on rural Interstates in Illinois.

Part 2 begins with a paper describing a new algorithm for use with image analysis to measure the number of pedestrians and their walking directions ( Lu et al.). Seneviratne and Javid discuss a statistical approach using Bayesian theory that combines available data, analysts' experience, and short or sample counts to estimate the expected pedestrian flow at a given site and time. The outcome is closer to the true mean flow and is suitable input to expansion models. Khisty describes an investigation into the use of simple non-Euclidean metrics for pedestrian and bikeway planning. Using the variables road pattern, road density, population density, size of green areas, and number of schools in the area, Al-Balbissi et al. related these to child pedestrian accidents in Zarqa, Jordan. The resulting models, which are based on multiple regression techniques, can give an estimate of the reduction in child accidents caused by some changes in road pattern and other variables.

Part 3 concerns motorist information systems. Hanscom evaluates the operational effectiveness of truck lane restrictions on multilane highways. The restrictions were applied using signing.

In work zones, Ogden et al. studied motorist comprehension of word and symbol signs. By means of personal interviews with 205 drivers, the authors identified comprehension problems with signing and other types of traffic control devices at an urban arterial work zone.
Different signal methods for controlling left-turning movements were compared by Hummer et al. Drivers were surveyed about comprehension of and preference for the various left-turn signals. Clear differences between the various techniques were found.

Anticipating information systems based on modern and emerging information technology, Koutsopoulos and Lotan propose a methodology for assessing the effectiveness of motorist
information systems in reducing recurrent traffic congestion. The method, based on stochastic traffic assignment models, can also relate the parameters of the problem, such as level and amount of information provided, percentage of users with access to information, and congestion levels. The method is demonstrated using a small suburban network.
In the final paper, Wenger et al. use an in-person survey to focus on (a) the behavior and decisions of commuters relative to their choice of route before departure; (b) the behavior and decisions of commuters while driving; and (c) the responses of commuters to manipulations of variable-message sign messages. From the survey, specific issues that need to be addressed in designing motorist information systems were identified.

# Performance Differences on Driving and Laboratory Tasks Between Drivers of Different Ages 

Thomas A. Ranney and Nathaniel H. Pulling


#### Abstract

A battery of closed-course driving and laboratory tests was developed for evaluating the skills required in routine suburban driving. Twenty-three younger (aged 30 to 51 years) and 21 older (aged 74 to 83 years) adults participated. Driving tests included responding to traffic signals, selecting routes, avoiding moving hazards, and judging narrow gaps. Laboratory tests included measures of perceptual style, selective attention, reaction time, visual acuity, perceptual speed, and risk-taking propensity. Older drivers were generally slower and less consistent in their driving. The groups did not differ from each other on measures of caution. In the laboratory, older drivers scored lower on tasks requiring rapid switching of attention. Differences in laboratory measures were larger, reflecting the greater difficulty of these tasks and the greater precision available in the laboratory. The pattern of greater variability of performance for the older drivers indicates that driving ability should not be judged on the basis of chronological age.


As the general population ages, the percentage of older drivers on the road is increasing. By the year 2020, an estimated 17 percent of the population will be 65 or older, resulting in more than 50 million older persons being eligible to drive (1). Furthermore, with the increasing trend toward suburbanization, people of ali ages, and especially older people, are becoming more dependent on their automobiles (2). At the same time, the total number of registered vehicles is increasing, commuting patterns are changing (3), and traffic congestion is becoming a major problem both in suburban and urban areas (4). Following recent advances in microelectronics, advanced technology is being investigated as a means of alleviating traffic congestion. Dynamic roadway signs with rapidly changing messages, together with in-vehicle communications systems, including cellular telephones and navigational aids with cathode-ray tube screens, are being combined to transform the driving task into a complex problem of information management. Whether the changing demands of driving combined with technological advances will be of special difficulty for aging drivers, either because of their inability to deal with complex information at a rapid rate or because of anxiety or feelings of alienation associated with their perceptions of the rapidly changing driving environment (5), will be of considerable interest in the near future, as the driving population continues to age. Therefore, the changing nature of the driving task and the implications of the changes for drivers of all ages should be examined.

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## BACKGROUND

## Accident Risk of Older Drivers

Until recently, older drivers were commonly accepted to have higher accident rates than middle-aged drivers ( $6-9$ ). This conclusion was generally based on the use of mileage as a measure of exposure to risk; accident rates for a particular age group were expressed as the number of accidents per miles traveled. Recently, however, people have argued that mileage-based rates are unreliable. One reason given is that mileage estimates have not been validated (10). A more common argument is that mileage rates may be misleading, because older people drive considerably less than younger people. According to Evans (11), "Greater than any increase in driver risk with increasing age is declining distance of driving." Because older drivers reduce their exposure to conditions of elevated risk and are thus less of a risk to other road users, Evans (11) concluded that "the problem of aging may thus be more one of reduced mobility than of reduced safety." However, the average number of miles driven annually by drivers 65 and older increased with each major survey taken between 1969 and 1983 (1), and such a compensatory tendency may well be specific to the current generation of older drivers. Because driving will have been a more pervasive and essential part of the daily life of the next generation of older drivers, they may be less willing to give up driving (1).

As an alternative to mileage-based accident rates, accidents per licensed driver within specified age groups have been reported. This accident rate may be more relevant for insurance purposes, for which the goal is to establish the accident risk for an individual in a given year. Yanik (10) reported that elderly drivers are underrepresented in accident frequencies relative to the number of licensed drivers within their age group. Specifically, although drivers over 65 make up 11.2 percent of the driving population, they are involved in 7 percent of all accidents. Cerrelli (12) reported that drivers over 75 have a crash involvement rate (per population) that is 2.5 times lower than that of drivers aged 40 and 5 times lower than that of 20 -year-old drivers.

## Specific Problems of Older Drivers

Older drivers are involved in different types of accidents than younger drivers. In industrial settings, older people tend to have accidents involving slowness in avoiding moving objects
or difficulty in recovering when thrown off balance. Road accidents are characterized by slowness in identifying and reacting to rapidly developing traffic situations (13). Complex traffic situations pose problems for the elderly driver. For drivers over age 50, the percentage of multiple-vehicle intersection accidents increases and the percentage of singlevehicle accident involvements decreases correspondingly. More than half the fatal accident involvements for 80 -year-old drivers occur at intersections, as compared to 25 percent or less for drivers up to 45 years old (14). Similar results have been reported by Waller et al. (8), who found that left turns were involved in about 25 percent of the crashes of drivers over 65 , almost twice the percentage of the average driver. Changing lanes, merging, and leaving from a parked position, i.e., driving situations that involve complex speed and distance judgments under time constraints, are more evident in the accidents of older drivers than in those of younger drivers. Brainn (15) reported that older drivers have difficulty with driving situations requiring backing. More recently, Monforton et al. (16) analyzed selected multivehicle accidents of American Automobile Association drivers in Michigan. They found accidents involving stop and yield signs to be more prominent for aging drivers. Rear-end and loss-of-control accidents were less frequent. Their analysis revealed a consistent trend toward increasing culpability with age for drivers over 55 years of age. These accident studies do not have exposure data that correspond to the accident factors; therefore, the reported overrepresentations reflect an unknown combination of risk and exposure factors.

Older and younger drivers differ in the types of precrash behaviors exhibited, and older drivers are cited for violating different traffic laws than younger drivers. Older drivers are more likely than younger drivers not to attempt an avoidance response before an accident. Sussman et al. (17) interpreted this as an indication of inattention. Planek and Fowler (18) reported the overinvolvement of older drivers in traffic "violations of omission," such as running red lights and stop signs and failing to yield. Drivers over 70 years old were convicted more frequently for sign, right-of-way, and turning violations, but less frequently for speed, equipment, and major violations (6).

Older drivers apparently compensate for age-related impairments by limiting their driving and avoiding risky situations and rush hours $(18,12)$. Whether this avoidance reflects increased concern about their capabilities or primarily the change in lifestyle that accompanies their withdrawal from the work force is unknown.

## Driving Competency

Reliable assessment of driving competency is critical to maintaining the mobility of aging drivers. Whereas drivers' accident histories are most often used for this purpose, their stability has been questioned. Miller and Schuster (19) found that past accident history is not a good predictor of future accident involvement and thus may not be a valid indicator of current or future driving competency. The need for an alternative measure of driving proficiency was asserted by McKenna et al. (20), who argued that detailed investigations of component skills would lead to better understanding of
differences in driving performance than conclusions based on a single criterion, such as accident rate. These authors concluded that "research should concentrate on specific skill deficiencies and their contribution to human error, rather than more immediately attempting to predict overall accident liability." In addition, practical considerations about the availability and quality of accident data, which rarely contain information in sufficient detail for research purposes, underscore the need for alternative ways of evaluating driving competency.

## Functional Age

An important aspect of the question of aging impairments is the age at which deterioration can be expected. Barrett et al. (21) concluded that information-processing ability begins to decline during the mid-40s. Results from other studies, such as that by Ponds et al. (22), suggest that an impairment related to dual-task performance occurs sometime after age 60. According to Willis (23), the age factor exhibits the widest of individual differences in pattern of decline, but most people will exhibit at least some age-related decline by age 80. A recent TRB study (1) identified age 75 as the point after which accumulated skills are offset by physiological and cognitive changes that accompany aging.

Clearly, as pointed out by Salthouse (24), individuals age at different rates. Furthermore, changes over the same number of years may have different meanings on different parts of the scale. For these reasons, the concept of functional age emerged. According to Kausler (7), functional age is the level of competence in basic skills that determine overall performance, in this case on-road driving skill. Early work on functional age attempted to develop a single index that could be used instead of chronological age. However, because aging is not unidimensional, a single measure cannot adequately represent the processes of aging. Subsequent work has therefore been undertaken to develop a functional age profile $(21,25)$. A profile allows an individual's position on a number of performance measures to be determined.

As implied by the functional age profile concept, different component skills can be expected to deteriorate at different rates for different individuals. Because aging cannot be characterized with a single index, decisions about driving competency cannot be made on the basis of chronological age. Despite increased concern about the practice of special testing for the elderly, the fact that such testing is being advocated is an admission that age per se does not cause increased accidents (7). Unfortunately, no functional age profiles of driving capabilities have yet been developed.

## METHOD

In response to the need for improved measures of driving competency, one objective of the Liberty Mutual automotive research program is the development of a safe-driving capability profile, which includes a battery of laboratory and driving tasks. Tasks were selected to assess the skills and abilities involved in everyday suburban driving. The driving tasks were implemented on a half-mile closed course that allowed drivers
to use their own vehicles. The use of drivers' own vehicles avoided problems of differential adaptation to unfamiliar research apparatus, such as simulators and instrumented vehicles.

The selection of specific driving tasks was based on a pilot evaluation of a number of tasks and reflected perceptions of the skills necessary for adapting to the changing suburban (i.e., relatively low-speed) driving environment together with the capabilities of the data collection facilities. Additional rationale, relating to the importance of decision making in driving and to the theories of driving behavior that motivated task selection, are presented elsewhere $(26,27)$. Laboratory tasks included measures of information processing, some of which have previously been shown to be related to accident rates (21). In the following paragraphs, the current battery of tasks will be described and the sensitivity of driving and laboratory measures for detecting performance differences between drivers of different ages will be evaluated.

## Subjects

Forty-four subjects ranging in age from 30 to 83 participated in driving and laboratory tests. The younger group ( 15 women and 8 men ) were 30 to 51 years old; the mean age was 39.7 years, and the standard deviation was 5.9 years. The older group ( 9 women and 12 men) were 74 to 83 years old; the mean age was 78.1 years, and the standard deviation was 3.1 years. Subjects were recruited with newspaper advertisements and from local senior citizen activity centers. All were active drivers. Subjects were paid $\$ 8.00$ to $\$ 10.00$ per hour for participation, depending on their responses to performance incentives.

## Apparatus

An instrumented driving range, including 0.5 mi of two-lane roadway, a signalized intersection, mobile hazards, and various regulatory and destination signs, was developed so that drivers could use their own vehicles. The instrumentation and its rationale are discussed by Ranney et al. (26). Traffic signal timing and data acquisition were controlled by a DEC PDP 11/23 computer in a van parked beside the intersection. Spot speed data were obtained from four pairs of inductive loops buried beneath the pavement. The pairs were separated by 36.6 m ( 120 ft ), with three pairs in front of and one pair beyond the intersection. Time of entry into the intersection was obtained from a single loop in each approach lane at the stop line. Traffic signal timing was related to the temporal position, that is, the time the vehicle would take to reach the intersection, which was computed using approach speeds. This computation compensated for differences in vehicle approach speed.

## Driving Test

The driving test consisted of three $30-\mathrm{min}$ trips. Each trip, composed of up to 20 laps of the closed course, required the driver to respond to a continuous sequence of driving situa-
tions. Primary tasks included responding to traffic signals with varied timing and selecting routes using information presented on traffic signs. A gap-acceptance task required drivers to select one of two routes at a junction on the course. One route was shorter, but it required drivers to drive through a gap formed by two construction barrels. Gap size changed and was determined by the width of each subject's vehicle. Based on pilot work, the following gap sizes were used: +3 , $-3,+6,-6,+9$, and -9 in. Drivers' judgments concerning the width of the gap and their willingness to attempt the gap were evaluated through this task.
Secondary tasks included avoiding unexpected moving hazards, such as a rolling ball or simulated baby stroller, responding to regulatory signs (speed limit and stop signs), and executing maneuvers created by cones and barrels. Drivers were instructed that they would be rewarded for safe driving and for completing each trip faster than a reference time. An experimenter accompanied the driver during instruction and training sessions. Subjects received practice until the experimenter felt that the instructions were understood. During the data collection, the experimenters were in or near the instrumentation van, alongside the intersection. Experimenters were careful not to distract the drivers during their approach to the intersection.

Both subjective ratings and objective measures of drivers' responses to the driving task situations were recorded. Drivers were rated on the following skills:

- Stop-and-go decision making,
- Gap judgment,
- Gap execution,
- Decision speed,
- Route selection,
- Speed maintenance,
- Vehicle control,
- Emergency hazard avoidance,
- Time to destination, and
- Ability to follow instructions.

Ratings were made on a 3-point scale. Drivers were observed by two or three raters, depending on staff availability. Ratings were discussed after each session, and a single consensus rating was recorded for each driver on each of the 10 categories. In cases where disagreement among the raters was not resolvable, a midpoint rating (e.g., 1.5 or 2.5 ) was recorded for the category. An overall rating of driving performance was computed as the average of the 10 categorical ratings.

Objective driving performance measures included the following:

1. Measures of intersection performance
-Stopping probability, the proportion of decision trials on which the driver stopped when faced with the yellow traffic signal (STOPPR);
-Stopping accuracy, the vehicle placement relative to the stopline on stopping trials (STPACC); and
-Intersection clearance margin, the mean difference between the time the vehicle exited the intersection and the onset of the red traffic signal (MARGIN).
'2. Measures of gap performance
-Number of attempts, the number of trials in which the driver attempted to drive through the gap (NOATT);
-Number of gap judgment errors, including selection of gaps too small and avoidance of gaps of equal or greater width (JUDGERR); and
-Number of gap execution errors, struck barrels or excessively slow speed (EXERR).
2. Speed measures
-Intersection approach speed, the mean speed over all trials (SPEED1), taken approximately 300 ft before the intersection, representing the speed before the traffic signal has changed from green to yellow;
-Intersection approach speed change, the mean over all trials (SPDDIF), measuring the influence of the traffic signal change on speed;

- Mean lap time, the mean time over all trials in one trip (LAPTIME); and
-Speed maintenance errors, instances of speeds over 35 mph (FAST35) or under 27 mph (SLOW) in approach to intersection.

4. Measures of vehicle control consistency

- Approach speed consistency, standard deviation over all trials of approach speed (SSPD1).

Measurement of route selection errors was eliminated because of insufficient data.

## Laboratory Tasks

Visual acuity (VISION) was measured with a standard Titmus tester, similar to those used for license renewal. Perceptual style was measured with the embedded figures test (EFT) (28). Perceptual speed was measured with three tests of the cognitive factors kit (29). The tests required a visual search for letters (VSEARCH), matching numbers (NUMBERS), and matching figures (FIGURES). The digit symbol substitution (DSS) test of the Wechsler adult intelligence scale is also a measure of perceptual speed and short-term memory, and has been used widely in studies of information processing and aging (30). Visual selective attention was measured with an analogue of the dichotic listening task developed by Avolio et al. (31). The total number of errors (VSATOT) and the number of switching errors (VSATSW) provided a measure of the efficiency of attention switching. The three measures of reaction time included simple (SRT), simple plus movement (MRT), and movement plus (two) choice (CRT) reaction times. Risk-taking propensity (RISK) was measured by the choice dilemmas questionnaire developed by Kogan and Wallach (32).

## RESULTS

Analyses were conducted to compare the performance of the younger drivers with that of the older drivers on driving and laboratory tasks. Analyses that examine the correlations between the laboratory and driving performance measures are presented elsewhere (27). Except for the subjective ratings, the TTEST procedure of SAS (33) was used to compare group variances for each measure and then to compute the appropriate $t$-tests to compare the two groups. The results for driving measures and laboratory measures are presented separately.

## Driving Performance

## Performance Ratings

Overall rated performance represents the most general measure of driving performance. Scores ranged from 1.3 to 2.6 . The mean rating for the drivers aged 30 to 51 was 2.21 , whereas the mean for drivers 74 to 83 years of age was 1.71. A MannWhitney U test revealed differences between the groups to be statistically significant ( $z=4.41, p=0.00003$ ). Differences between the two groups for each of the 10 categorical ratings making up the overall rating are presented in Table 1. Differences between the two age groups were largest for the following four rating categories: decision speed, gap execution, route selection, and comprehension of task instructions. Smallest differences were apparent for emergency response to moving hazards and speed maintenance.

## Intersection Performance

Differences between the two age groups on three measures of intersection performance are presented in Table 2. On the basis of group means, the older drivers were slightly more likely to stop (STOPPR) when faced with a yellow traffic signal than were the younger drivers ( 0.50 versus 0.36 ). This difference, however, was not statistically significant. The groups did not differ in the accuracy of stopping (STPACC), which represents the position of the vehicle relative to the stop line on stopping trials. This measure reflects vehicle control in stopping. On trials in which the driver did not stop, the clearance margin (MARGIN) represents the time between the red onset and the time at which the vehicle exited the intersection. The negative clearance margins indicate that vehicles exited before the red onset. Positive margins indicate that the vehicle was still in the intersection as the light turned to red. The group means did not differ; however, the older drivers exhibited a higher variance level than the younger drivers on this measure.

## Gap Performance

Differences between the two groups in three measures of gap performance and the results of statistical tests are also presented in Table 2. No difference was observed in the number of gaps attempted (NOATT). For this measure, the level of variability associated with the older group was higher than that for the younger group. The older drivers were more likely to make judgment errors (JUDGERR) with regard to gap size. Again, the level of variability was higher for the older group. The greater likelihood of older drivers to make gap execution errors (EXERR) was also statistically significant, as were differences in group variances.

## Speed Measures

Results of speed measures analyses are also presented in Table 2. Intersection approach speeds, which reflect vehicle speed before the traffic signal change, were not different for the two age groups. Mean lap times, which reflect speed over the

TABLE 1 AGE GROUP DIFFERENCES ON RATED DRIVING CATEGORIES

|  | Mean <br> Younger | Mean Older | $\begin{gathered} \% \\ \operatorname{diff} f^{1} \\ \hline \end{gathered}$ | $\text { Significance }{ }^{2}$ |
| :---: | :---: | :---: | :---: | :---: |
| Stop/go decision | 1.3 | 1.1 | 15\% | . 09 |
| Route selection | 2.3 | 1.7 | 26\% | . 0006 |
| Decision speed | 2.5 | 1.6 | 36\% | . 0004 |
| Emergency response | 2.1 | 2.0 | 5\% | . 36 |
| Gap judgment | 2.1 | 1.6 | 24\% | . 012 |
| Gap execution | 2.5 | 1.7 | 32\% | . 0005 |
| Vehicle control | 2.4 | 1.9 | 21\% | . 006 |
| Speed maintenance | 2.1 | 1.8 | $14 \%$ | . 28 |
| Time to destination | 2.3 | 1.9 | 17\% | . 04 |
| Comprehension | 2.5 | 1.9 | $24 \%$ | . 0014 |
| 1 Computed as the difference between the two groups as a percentage of the mean for the younger group |  |  |  |  |
| $2 \mathrm{p}>\|z\|$, two-tailed significance probability, based on |  |  |  |  |

TABLE 2 AGE GROUP DIFFERENCES ON DRIVING MEASURES

|  | Means |  |  | Statistical Significance |  |  |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: |
|  | Youn | ger(sd) | Olde | r(sd) | Means ${ }^{1}$ | Variances ${ }^{2}$ |
| STOPPR | . 36 | (.22) | . 50 | (.29) | . 07 | . 22 |
| StPACC | 128.7 | (10.4) | 129.0 | (12.6) | . 93 | . 38 |
| MARGIN | -. 10 | (.22) | -. 23 | (.50) | . 29 | . 0004 |
| NOATT | 10.13 | (2.12) | 9.24 | (4.36) | . 40 | . 001 |
| JUDGERR | 3.61 | (1.16) | 4.76 | (2.36) | . 05 | . 002 |
| EXERR | 0.48 | (.99) | 1.38 | (1.56) | . 03 | . 04 |
| SPEED1 | 29.09 | (1.85) | 28.52 | (2.72) | . 42 | . 08 |
| SPDDIF | 1.89 | (1.49) | 2.74 | (1.71) | . 09 | . 53 |
| LAPTIME | 90.46 | (8.75) | 100.76 | (12.17) | . 003 | . 15 |
| FAST35 | 1.87 | (4.37) | 2.86 | (6.03) | . 53 | . 15 |
| SLOW | 17.43 | (23.38) | 30.38 | (33.13) | . 14 | . 11 |
| SSPEED | 1.40 | (.34) | 1.79 | (.40) | . 001 | . 50 |

$1 \mathrm{p}>|t|$, two-tailed significance probability
$2 p>F^{\prime}$, where $F^{\prime}$ is the ratio of the larger to the smaller group variance
entire course, were considerably slower for the older drivers. The differences between the age groups in the maximum speed change in the intersection approach were not statistically significant. Similarly, differences between the groups in the frequency of exceeding 35 mph or driving more slowly than 27 mph in the intersection approach were not statistically significant. The standard deviation of intersection approach speed was computed for each driver as a measure of vehicle control consistency. The large difference indicates that the older drivers were considerably less consistent in their approach speeds than were the younger drivers.

## Laboratory Performance

Results of analyses for laboratory measures are presented in Table 3. With the exception of the risk-propensity questionnaire (RISK), all laboratory measures exhibited significant differences between the two age groups at the $p<0.05$ level. The two age groups' differences in visual acuity (VISION) were apparent. In addition, differences were largest for the EFT, the visual selective attention tests (VSATOT, VSATSW), the figure matching test of perceptual speed (FIGURES), and the DSS task. With the exception of the EFT, these tasks require rapid switching of attention between two or more sources of information. Smaller differences were apparent for the three measures of reaction time (SRT, MRT, CRT) and for the visual search for letters (VSEARCH) and number matching tasks (NUMBERS).

## DISCUSSION OF RESULTS

One objective of the current analysis was to determine the sensitivity of the performance measures for detecting impairment effects associated with aging. The selection of drivers between 74 and 83 for the older group was intended to maximize the likelihood that at least some age-related deterioration would be available for detection. The pattern of observed differences, all reflecting the poorer performance of the older group, indicates that both the driving and the laboratory measures are sensitive to age-related performance differences. Because this study is cross sectional, however, differences cannot be directly attributed to the effects of aging. Rather, differences may suggest effects of aging, but they more accurately reflect differences between drivers of different ages in the current driving population. Furthermore, the use of volunteers and recruits from senior citizen activity centers may lead to questions about how representative the sample was. The sample of younger drivers was probably fairly representative of 30 - to 50 -year-old drivers in this area; however, the difficulty of recruiting older drivers and the number of referrals who declined to participate indicate that the older drivers were probably better than average for their age group. Observed differences between the two age groups may, therefore, understate actual differences in the general driving population.

Overall, the older drivers were given lower ratings than the younger drivers. This lower rating reflects a number of differences, including slower decision speed; errors in route selection, gap execution, and vehicle control; and difficulty

TABLE 3 AGE GROUP DIFFERENCES ON LABORATORY MEASURES

|  | Means S |  |  | Statistical Significance |  |
| :---: | :---: | :---: | :---: | :---: | :---: |
|  | Younger (sd) | Older | (sd) | $\text { Means }{ }^{1}$ | Variances ${ }^{2}$ |
| EFT | 62.2 (34.5) | 108.5 | (42.3) | . 0005 | . 38 |
| DSS | 67.7 (10.1) | 45.7 | (12.7) | . 0001 | . 32 |
| VSEARCH | 30.3 (8.4) | 25.0 | (6.8) | . 03 | . 38 |
| NUMBERS | 12.45 (3.08) | 9.88 | (2.16) | . 04 | . 12 |
| FIGURES | 33.83 (6.73) | 20.36 | (7.32) | . 0001 | . 72 |
| RISK | 76.2 (12.9) | 82.7 | (14.5) | . 15 | . 63 |
| VSATOT | 136.8 (37.2) | 190.2 | (21.7) | . 0001 | . 03 |
| VSATSW | 62.56 (22.0) | 87.63 | (11.2) | . 0003 | . 008 |
| VISION | 20.85 (7.77) | 40.35 | (22.76) | . 001 | . 0001 |
| SRT | . 31 (.04) | . 36 | (.07) | . 007 | . 04 |
| MRT | . 60 (.08) |  | (.19) | . 03 | . 0004 |
| CRT | . 68 (.09) | . 81 | (.22) | . 02 | . 0006 |

[^1]understanding task instructions. No differences between the two groups were observed in drivers' responses to simulated emergency situations.

With regard to measured driving performance, the older drivers were slower overall, as evidenced by longer lap times. Poorer vehicle control for the older drivers was evidenced by differences in gap execution, whereas less consistent vehicle control was indicated by greater approach speed variability. A number of apparently meaningful differences did not reach statistical significance because of the large individual differences between drivers in the older group. This pattern was most evident for measures associated with the gap task, all of which exhibited greater variability for the older group. This finding is consistent with previous work, which indicated that considerable variability exists in the rate at which age-related changes appear (23), and underscores the importance of not judging driving ability on the basis of chronological age.

In general, performance on the laboratory tests revealed larger differences than were evident on the driving tasks. These differences were caused by the greater difficulty of some of the laboratory tasks, most notably the visual selective attention task, and the greater precision of measurement available in the laboratory. In the laboratory, the rates for the more complex tasks, which required use of short-term memory and switching of attention between two sources, exhibited larger differences than the more simple tasks. In contrast to patterns for other measures, results on the visual selective attention task revealed larger variances for the younger group than for the older group. The task was so difficult for the older group that virtually all older drivers were unable to perform the majority of the tasks. Variances for the three reaction time measures revealed the more common pattern of greater variability for the older group. To the extent that the consistent differences between the two groups suggest an overall decline on all information-processing abilities, rather than selective differences, the results are consistent with those reported by Panek et al. (34).
Several of the analyses provided information about whether older drivers are more cautious than younger drivers. The choice dilemmas questionnaire has been used as a measure of risk-taking propensity, and age effects have been reported in previous work (35). The current results revealed no difference between the two groups on this measure. In addition, two performance measures, the number of gaps attempted and the proportion of stops at the traffic signal when faced with a yellow light, provided direct measures of drivers' willingness to take risks. The older drivers were not less inclined to attempt the gap task, but they were slightly (although not significantly) less likely to stop at the traffic signal. Together, these results indicate that the older drivers were not more cautious than the younger drivers.

The current development followed, in part, the theoretical work of Barrett et al. (36), whose specification of relevant driving skills was based on accident data collected before and during the 1970s. The changing nature of both the driving environment and driver population make it likely that different types of errors are currently involved in accident causation. Accordingly, analysis of more current accident data, focusing on the behavioral errors related to accident causation, will be necessary for development of a contemporary model of driving behavior, on which a comprehensive test
battery can be based. Compromise will always be necessary in implementing such a test battery, because of constraints imposed by simulation capabilities and safety considerations. Validation will also be difficult, because of the questionable suitability of past accident rates as the criterion, and may ultimately require a longitudinal study, where future accident involvement can be predicted by driving test performance. Nevertheless, the present results reveal age-related differences in both laboratory and driving performance and represent the first step toward the development of an assessment tool for use with drivers of all ages. Although previous research documents performance differences on laboratory tasks, few previous studies have examined age-related performance differences using actual driving tasks.

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# T-Intersection Simulator Performance of Drivers with Physical Limitations 

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#### Abstract

The effect of restricted head and neck movement on driving performance was measured by decision time at simulated T-intersections. Little traffic safety and human factors literature concerning the performance of drivers with physical limitations was found. Most accidents involving drivers with diminished capacities occur at intersections; therefore, simulated T-intersections were selected for study. A laboratory study using a driving simulator was selected to provide better experimental control, safety, and repeatability. A fixed-base driving simulator that incorporated videorecordings of intersections to provide a 180 -degree field of view was used. Eighteen intersections with various levels of traffic volume and sight distance were studied. The 72 test subjects were either between the ages of 30 and 50 or between 60 and 80 , and half in each group had a restricted range of neck movement. The subjects' task was to depress the brake pedal, watch the video presentations of the T-intersections on three screens, and release the brake pedal when it was safe to make a left turn. The decision time was a measure of driving performance. The following hypotheses were confirmed: (a) decision time increases with age, and age effects dominated the other factors studied; (b) decision time increases with age and level of impairment, indicating that younger drivers are able to compensate for their impairments, but older drivers both with and without impairments are unable to make compensations in their driving performance; and (c) skewed intersections are hazardous for drivers with neck impairments. Further laboratory and field studies are recommended to validate the study results and to examine the problem of skewed intersections.


A study of the performance of drivers with physical limitations was undertaken at the Turner Fairbank Highway Research Center of FHWA, U.S. Department of Transportation, in McLean, Virginia, as part of the graduate research fellowship program. A review of the current transportation literature showed that little is known about the relationship of biomechanics and driving performance. A driving simulator was used to examine the relationship between head and neck mobility and decision time at simulated T-intersections.

## BACKGROUND

Accident rate statistics document the increase in accident rates of older drivers and drivers with diminished capacities on a miles-driven basis and indicate the need to study the performance of these drivers (1). The population demographics predict a dramatic increase in the percentage of older people in the total population by the year 2000 and beyond (2). The majority of accidents involving the older driver occur at intersections, during lane changing and turning maneuvers (3).

[^2]These facts indicated the need to study the performance of drivers with physical limitations at intersections.

The problems of drivers with diminished capacities need to be understood to determine safer road design standards and operational and control strategies. A better understanding of these drivers' characteristics will facilitate the design of education programs to help these drivers compensate for their limitations. Human factors, vehicle characteristics, and road geometric requirements form the basis of most of the standards used for the design of highways and streets. The study of drivers with diminished capacities at intersections covers a broad spectrum of fields, from human factors, gerontology, ophthalmology, and ergonomics to traffic engineering. Significant studies that examine the psychological and cognitive aspects of drivers with diminished capacities have been undertaken. These studies are complementary to studies of physical limitations (4).

The transportation and human factors literature indicated that little is known about the relationship between physical limitations and driving performance; therefore, this relationship was selected for study. Most of the human factors literature relating to older drivers has focused on the visual and cognitive aspects of driver behavior and performance. The gerontology and human factors literature $(5-7)$ has clearly documented the changes in visual acuity and accommodation with age. Drivers with diminished capacities have different sensory, cognitive, and physical thresholds than other members of the driving public (8). These drivers require more stimuli for perception, or extra time to react, as a result of physical limitations. Several reports $(9,10)$ have shown that a physically challenged individual's driving performance, as judged by accident statistics, is normally average or above average. However, the performance of drivers who are marginally physically impaired, either permanently or temporarily, has not been studied.

Intersections require drivers to make decisions about turning or crossing and present conflicting traffic flows and changing roadway geometrics, which increase driver workload. Increased accident rates at intersections appear to be related to the implementation of new traffic control devices, high traffic volumes, and low sight distance (11-13). Polus (14) points out that more restrictive signalization does not necessarily result in a decrease in accidents or unsafe movements.

## RESEARCH OBJECTIVES

A better understanding of the effects of physical limitations on driving behavior and decision-making ability was sought.

The behavior of drivers at simulated T-intersections was investigated to determine the relationships between the range of movement of the head and neck, the visual field, and the decision time for a simulated traffic maneuver.

## EXPERIMENTAL DESIGN

A driving simulator in a laboratory environment was used, because of the hazards and lack of experimental control of a field study. Three rear projection screens were used to provide a 180 -degree field of view in the driving simulator. This method of providing the drivers' perspective of the roadway was more realistic than the other methods of intersection simulation. The performance times of drivers with physical limitations at simulated unsignalized T -intersections were examined to determine the relationship between physical limitations of the neck and decision time. Each intersection presented different geometrics and traffic volumes.

The experiment was a 2 (age) $\times 2$ (impairment) $\times 3$ (sight distance) $\times 2$ (volume) factorial design, with repeated measures on sight distance and volume. The subjects were partitioned according to age and impairment, and two levels of traffic volume and three levels of restricted sight distance were established. The independent variables were the age and impairment of subjects.

The subjects were divided as follows:

- 30 to 50 years, impaired: 15 subjects;
- 30 to 50 years, unimpaired: 15 subjects;
- 60 to 80 years, impaired: 15 subjects; and
- 60 to 80 years, unimpaired: 15 subjects.

Figure 1 is a histogram of the distribution of ages in the two groups (15). The median age for the 30 - to 50 -year age
group was 40 years, and the median age for the 60 - to $80-$ year age group was 67 . For this research, impairment was defined by a combined static range of movement of the head and visual field of less than 285 degrees. A range from 285 to 360 degrees was defined as no impairment. No definitive definition of impairment was found in the literature, so the choice of 285 degrees was based on the functional requirements for driving.

The 18 intersections had variations in traffic volume and sight distance. The two levels of traffic volumes were measured in terms of average gap length ( $g$ ). The videotaping was done during morning rush hour for some of the intersections and at midday for the others. The gap lengths in the cross-stream traffic on the videotapes were measured to determine the traffic volume for each intersection. Light traffic volumes consisted of gap lengths of 8 sec or longer on both traffic streams. Moderate traffic volume had gaps of less than 8 sec in both traffic streams. Nine intersections had low traffic volumes, and nine had moderate traffic volumes. Of the sight distances for the 18 intersections, six were below standard, six were approximately at standard, and six were longer than standard. The sight distance standard was defined by AASHTO's Policy on Geometric Design of Highways and Streets (16). The intersections were all within a $5-\mathrm{mi}$ radius of the Turner Fairbank Highway Research Center in McLean, Virginia. The terrain is rolling, so few of the intersections were level. All but one were 90 -degree intersections. All intersections were filmed in daylight, when the pavement was dry.

The independent variables were age, impairment level, traffic volume level, and intersection sight distance. The measured or dependent variables were as follows:

1. Response time, which was determined by measuring the time between the tone indicating that an intersection pres-


FIGURE 1 Histogram of subjects' ages.
entation had begun and the moment the brake pedal was released in preparation for a left turn;
2. Static range of motion (the principal measure of impairment), which was the maximum head turn angle of each subject as measured by the goniometer before testing; and
3. Visual field, which was the maximum field-of-vision width of each subject, measured on the ortho rater before testing.

Response time was the principal dependent variable for the research.

## Subjects

Participants between the ages of 30 and 50 , or 60 and 80 , were involved. Approximately one-half the subjects in each age group had some degree of physical limitation, which restricted the range of movement of their heads and necks but was not severe enough to require major vehicle modifications, such as additional mirrors. Subjects were recruited as paid participants through local advertisements and through contacts with local agencies, such as the Arthritis Foundation and the American Association of Retired Persons. Each participant was compensated $\$ 25.00$ for involvement in the study. All participants were required to have a valid driver's license and to drive an average of at least $10 \mathrm{mi} /$ week.

The subjects exhibited a wide variety of driving behavior and physical skills. Many subjects who thought that they were not impaired had less than a 105-degree static range of neck movement, and others who had arthritis and thought they were impaired showed no impairment in range of neck movement. Many of the subjects with arthritis were taking antiinflammatory medication and participated regularly in exercise programs sponsored by the Arthritis Foundation. In the 60 - to 80 -year age group, nearly all the subjects showed limited neck mobility. Driving skills in this age group also varied greatly. Some of the variability could be explained by the type of vehicle regularly driven, lifestyle, and attitude. In general, the female subjects in both groups were much more cautious and required many more practice intersections before they felt confidence to proceed with the 18 test intersections. Videotapes of two extra intersections were used for practice. The practice intersections had moderate traffic volumes and mixed sight distance. Most of the male subjects only required two practice intersections. Twelve of the subjects missed four or more intersections; therefore, the final statistical analysis was performed using the data from 60 subjects. A missed intersection resulted from the subject removing his or her foot from the brake before the sound of the tone marking the beginning of the measure of response time. As a result, no data were collected for that subject at that intersection.

## Experimental Procedure

The participants were screened over the telephone to determine whether or not they met the criteria for participation, as well as to explain the general nature of the research and their participation. At the beginning of the experiment, the general purpose of the research was outlined in the instruction sheets, and each participant was asked to sign an informed
consent form. The informed consent form is standard policy at the Turner Fairbank Highway Research Center. After the introduction, the following information was collected as part of the experimental design: participant's age, sex, description of physical impairment for the subjects in the impaired group, whether they wear glasses for driving, static range of head and neck movement, and visual field.

The questioning was followed by a range-of-movement test of the neck and head, and a visual field test. The initial procedures took approximately 30 min to complete. Afterwards, the participants were allowed a short break. The participants were then introduced to the simulator equipment and permitted a few minutes to become accustomed to the equipment. Participants were also given instructions for the test and permitted to ask any questions concerning the test procedures or equipment.

The subjects' task was to watch video presentations of the intersections on the three rear projection screens. Before each intersection was presented, the intersection was announced and the subject depressed the brake pedal. A few seconds of run-in of the scene followed, then the audible beep signalled the subject that decision timing was beginning. The subjects watched the scene. When they felt it was safe, they would indicate that they were ready to make a left turn by releasing the brake pedal. The release of the brake pedal would signal the end of that intersection's presentation, and a pause of 1 to 2 min would take place before the presentation of the next intersection. The videotapes covered half of the visual range; therefore, the test subjects had to mentally fill in the visual image between the screens. The test subjects had to judge when there were acceptable gaps in both the left and right traffic streams. In general, the traffic volumes were low in one or both traffic streams.

Two trial intersections were presented for practice, often repeatedly. Then 18 test intersections were presented, sequentially and without repetition, to each participant. For some participants, subjective responses to each intersection were made during a short period after the presentation of each intersection. Response information was recorded on a data sheet and on a data acquisition system for analysis at a later time. The data acquisition system recorded the response time and the degree of head movement. After the presentation of the final intersection, each participant was debriefed and paid.

## EXPERIMENTAL RESULTS

The experimental results were split into two segments: totals (across all subjects and all intersections) and individual intersection summaries (across all subjects). The statistical results of the intersection summaries were similar to the statistical results for the totals of all subjects at all intersections. More error was introduced into the totals, because the decision time was averaged only over the correctly answered intersections for each subject, and each intersection had a different time interval depending on traffic volume and geometrics. Many of the older subjects did not follow instructions correctly and misjudged several intersections. As a result, the data could not be used for those intersections, and the intersections were judged to be incorrect. Therefore, the totals results only
represent the correct intersections driven by each subject. This process introduces bias, because some intersections were more consistently incorrect than others. Individual intersection statistics were examined to eliminate errors resulting from different intersection time intervals, sight distance, volumes, and geometrics.

The totals summary statistics give a general impression of the significant relationships. Subjects' ages were coded into two groups; subjects 30 to 50 years old were in Group 1, and subjects 60 to 80 years old were in Group 2. The ANOVA for the relationship of average decision time versus age was significant at the 4 percent level. The means and standard deviations of average decision time versus age are presented in Table 1. A smaller left-turn decision time indicates better driving performance, because the driver has more of the gap time to accelerate to speed and will not affect the uniform speed of the traffic stream. The older drivers took 2 sec longer to decide to turn at T-intersections than the younger drivers. The standard deviations for the older drivers are 0.43 sec higher than those of younger drivers, which indicates greater inconsistency in this segment of the population.

The ANOVA indicates that the relationship between average decision time and functional level is significant at the 8 percent level. The definition of functional level is a combination of age and impairment level:

- Functional Level $1=30$ to 50 years old with no impairment,
- Functional Level $2=30$ to 50 years old with impairment,
- Functional Level $3=60$ to 80 years old with no impairment, and
- Functional Level $4=60$ to 80 years old with impairment.

The means and standard deviations of average decision time versus functional level are presented in Table 2.

The mean decision time increases with functional level, and the increase in decision time between the younger and older age groups is approximately 2 sec . The $1.25-\mathrm{sec}$ increase in standard deviation with impairment can be explained by the wide diversity of impairment level and the less consistent driving behavior in this subject group. The implication of

TABLE 1 AVERAGE DECISION TIME IN SECONDS VERSUS AGE $(p=0.04$, COEFFICIENT OF VARIATION $=29.88$ )

| AGE | MEAN | STD.DEV |
| :---: | :---: | :---: |
| $30-50$ | 11.3 | 3.45 |
| $60-80$ | 13.3 | 3.88 |

TABLE 2 AVERAGE DECISION TIME IN SECONDS VERSUS FUNCTIONAL LEVEL ( $p=0.08$, COEFFICIENT OF VARIATION $=29.66$ )

| FUNCTIONAL LEVEL | MEAN | STD.DEV. |
| :--- | :---: | :---: |
| $130-50$, UNIMPAIRED | 11.3 | 2.87 |
| $230-50$, IMPAIRED | 11.4 | 4.09 |
| $360-80$, UNIMPAIRED | 12.1 | 3.08 |
| $460-80$, IMPAIRED | 14.4 | 4.35 |

these results is that the younger impaired drivers were able to compensate for their impairment in their driving behavior but the older drivers; either impaired or unimpaired, were unable to make the necessary compensations. The older drivers took longer to make a decision and were more inconsistent in their decision making than the younger drivers. The larger standard deviation is a measure of the inconsistency in driver behavior. Both the longer decision time and the inconsistency of the older drivers support the hypothesis that reaction times are influenced by age. These conclusions are also suggested in the literature and highway accident data $(1,17,18)$.

## EXPERIMENTAL OBSERVATIONS

The experimental observations contained qualitative information gathered during conversations with the subjects, as well as observations made during the data collection phase. The observations give further insight into the problems of drivers with diminished capacities at T-intersections.

Several bad intersections are near the Turner Fairbank Highway Research Center, prompting many of the subjects to comment on intersection design in general. Several of the older subjects mentioned that they had problems with skewed intersections (not 90 degrees), because they were forced to turn their heads and look over their shoulders, which was painful or not possible. Some of these participants mentioned that they would drive out of their way to avoid skewed intersections, because they could not turn their head enough to judge gap length and approaching vehicle speed. These people were asked if they had any problems with merging on freeways and highways. The most frequent response was that they could always look ahead, and use their rear view mirrors or side mirrors when they were in the merge lane. The skewed intersection presents a greater problem because the vehicle is stopped and a greater gap length is needed for acceleration to speed. Skewed intersections are often complicated by poor sight distance conditions associated with the terrain or foliage. Hauer (13) discusses the problems of intersection angles of 75 degrees or less and states, "The need of extensive head movement is in itself a problem for the older segment of the driving population, which may not have been taken into account in AASHTO's geometric design policy." These comments support the observations made by the test subjects.

Subjects from both age groups mentioned that they often felt that they had problems with gap length judgment and sight distance because of obstructions such as utility poles, street signs, or foliage. These comments are consistent with the literature $(13,19)$.

## CONCLUSIONS AND RECOMMENDATIONS

In general, older impaired drivers require more time to perceive and react to traffic conditions at T-intersections. The major specific conclusions of the study are as follows:

1. Older drivers take longer to decide to make left turns at simulated T-intersections, and their driving behavior is much more inconsistent than that of younger drivers.
2. The relationship between decision time and functional level, which is a combination of age and impairment, shows
that younger drivers with impairments are able to compensate for their impairments in their driving behavior. Older drivers, either with or without impairments, are not able to make the same compensations in their driving behavior.

The conclusion that older drivers require more time to perceive and react to traffic conditions at T-intersections has two main traffic safety implications:

1. Older drivers need to change their driving behavior to account for the changes in their reaction time. Special driver education courses exist to help mature drivers learn more about their own driving needs, as well as to account for changes in traffic and roadway design. Better incentives for mature driver education, such as lower insurance rates and easier license renewal procedures, would encourage more older drivers to participate in driver improvement programs.
2. Traffic engineers need to account for drivers with diminished capacities in the design of intersections and roadways. The perception reaction time factor in the sight distance calculation should be increased, particularly at complex intersections.

An increase in average decision time with age, and also with age and impairment level, has been shown. These findings are based on a laboratory study; however, further studies in the laboratory and the field are required to validate the results. The experimental observations indicated skewed intersections present a significant problem to drivers with limited neck movement and should be studied further.

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# Behavioral Model of Freeway Exiting 

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#### Abstract

A general model of diverging from a freeway to an exiting area was developed on the basis of purely behavioral considerations. The model proposes three sequential response elements that together define speed-change lane length. The first element is a criterion for a driver to diverge from the freeway on the basis of the angular velocity of the exit ramp gore (relative to the driver while approaching the exit in the right lane of the freeway). The second element is the distance required for drivers to complete a steering-control maneuver onto the speed-change lane. The third element is the distance at which drivers begin to brake in order to move from tracking the speed-change lane to tracking the curve of the exit ramp. A mathematical definition of each element was developed. These definitions allowed the prediction of each of the critical distances. The predicted distances were compared to observed exiting behavior on both curved and tangent exit ramps. The diverge, steering-control, and begin-braking distances observed on curved ramps were not statistically different from those predicted by the model. The begin-braking distance estimate for tangent exit ramps was not validated, probably because of instrumental errors in the field data analysis. The results indicate that the model is a reasonable representation of the exiting maneuver and provides a rational means for the design of exit speed-change lanes in terms of its ability to predict how far upstream from the wedge point the speed-change lane should begin, how far from the wedge point the steering-control maneuver ends, and how far from the wedge point the driver will initiate braking for a curved ramp.


To exit a freeway in a safe and efficient manner, the freeway driver must navigate either an off-ramp junction or a weaving area (1). Ramp junctions are critical areas on the freeway mainline in terms of highway safety. A basic freeway segment with the same length and volume as a section in a ramp junction will have fewer accidents, on average. This situation suggests that the current design procedure (2) used for these junctions does not optimally meet the requirements of drivers. A model based on the process the driver performs to exit the freeway was developed. Such a validated model can provide a more rational means for the design of speed-change lanes (SCLs) in off-ramp junctions.

## EXITING PROCESS

To exit the freeway by an off-ramp junction, the driver must perform at least four tasks: (a) detecting the existence of the

[^3]deceleration lane; (b) diverging from the mainline traffic stream onto the SCL, that is, initiating and performing a steering maneuver; (c) completing the steering maneuver and reorienting to tracking the tangent speed-change lane; and (d) introducing deceleration or steering-control response to the exit ramp's controlling point. The first step involves the sighting of the off-ramp junction from a distance as the driver travels on the freeway. The driver may become cognizant of the exit before it is visible because of informational signs or previous experience. This knowledge should lead the exiting driver to move to the right lane of the freeway. At some point, the exit lane will become visible, and for all practical purposes will be perceived as an expansion of the visual angle subtended by the additional lane. Given normal human visual acuity, this perception should occur long before the physical beginning of the SCL. Both signs and the lane addition may serve as alerting stimuli, but they do not provide a criterion for the initiation of the diverge maneuver.

When the vehicle is in linear motion on the freeway, the driver's field of view is in a continuous state of change. However, certain principles of motion perception are applicable. Roadway elements on which the driver focuses in the far distance appear stationary, whereas points in the near distance and off the line of regard appear to be in motion (3). These objects form a continuum, which has a transition distance at which specific points of focus change from their stationary state to a state of motion. This continuum is defined in terms of the angular velocity of elements within the visual field. The points of focus in the transition zone have an angular velocity at or near the driver's angular velocity threshold $\omega_{r}$. An $\omega_{t}$ value of $0.004 \mathrm{rad} / \mathrm{sec}$ is applicable for most drivers (4). The distance at which this transition occurs is called the forward reference distance $S$, which is a function of speed. When elements at the driver's point of focus have an angular velocity $\omega$ that is less than the angular velocity threshold, the points appear stationary. However, when $\omega \geq \omega_{t}$, any point off the line of regard has a detectable component of lateral motion.

## Detecting the Movement of the Deceleration Lane

If a driver plans to exit the freeway from a right-hand offramp junction, the driver's vehicle is assumed to be in the right-hand freeway lane for some distance before the SCL. As the driver approaches, the exit gore area elements will increase in angular velocity until they reach the driver's angular velocity threshold. Assuming that the driver's point of focus constantly varies, loci at the beginning of the exit ramp will eventually be scanned, especially if the driver plans to exit. At some point, the mouth of the exit ramp will attain a
suprathreshold angular velocity (i.e., the ramp will be perceived to move laterally relative to the driver). Thus, the ramp appears to change from being a stationary locus to a moving one.

## Diverge Criterion

Detection of the angular velocity of the points near the beginning of the exit ramp is hypothesized to be the cue for the exiting driver to diverge. This criterion initiates a steeringcontrol maneuver onto the SCL. The driver is not responding to the magnitude of target size or target angle per se; rather, the driver is responding to the changes in target size and angle (i.e., angular velocity) caused by the forward motion of the car. This proposed diverse criterion will define the distance at which a driver will normally begin a steering change from freeway to SCL. If this hypothesis is valid, then it will define the minimum speed-change lane length (SCLL).

## Steering-Control Response

On meeting the diverse criterion, the driver will introduce a steering input to move from the freeway to the SCL. This process requires a distance determined by the acceptable yaw velocity a driver will tolerate. Previous research (5) has evaluated this process and defined it explicitly. The chosen distance defines where a driver will be located on the SCL relative to the ramp connector. Obviously, this point should be reached before a driver is required to respond to the ramp geometry.

## Brake Deceleration Criterion

Once drivers complete the transition to compensatory tracking of the SCL, they must prepare for a steering or braking response to the exit ramp geometry. For curved exit ramps, braking is hypothesized to be initiated when the inner edge of the controlling curve enters the forward reference distance $S$, with a braking angular velocity $\omega_{b}$ in the range of 0.1 to $0.3 \mathrm{rad} / \mathrm{sec}$. The value of $0.1 \mathrm{rad} / \mathrm{sec}$ is used as the default value for $\omega_{b}$ because that is the value at which changes in angular acceleration approach a minimum (3), as shown in Figure 1. This distance $\left(L_{B_{c}}\right)$ would represent the transition from compensatory to pursuit tracking.

For tangent exit ramps (e.g., ramps in diamond interchanges), braking is hypothesized to occur when the points near the ramp terminus reach the driver's angular velocity threshold $\omega_{\text {, }}$ at the forward reference distance. Examples of points near a ramp terminus are a traffic signal, a stop sign, or the rear of a vehicle at the end of the queue.

## MATHEMATICAL MODEL FOR THE EXIT MANEUVER

On the basis of this description of the diverge process, the SCL was divided into three general longitudinal segments, as shown in Figure 2. The first segment is the steering-control


FIGURE 1 Angular acceleration versus angular velocity.


FIGURE 2 Three length components.
length $L_{\mathrm{SC}}$. The second segment is the deceleration-in-gear length $L_{G}$, and the third segment is the braking distance $L_{B}$. The sum of the lengths of the three segments is equal to the desirable SCLL.

## Divergence from Freeway to SCL

As a driver approaches an exit, the individual needs only momentarily view the exit gore, as shown in Figure 3. Point $P$ is the location at which the exit gore will generate an angular velocity greater than the threshold, $\omega_{r}$. This position is the criterion for the exiting driver to initiate the steering-control maneuver from the right freeway lane onto the SCL. The distance, $L_{\text {Det }}$, of Point $P$ from the wedge point is defined by the following equation:
$L_{\text {DetI }}=\left[\frac{V_{d}}{\omega_{t}}\left(h_{1}+y^{\prime}\right)-\left(h_{1}+y^{\prime}\right)^{2}\right]^{1 / 2}-\frac{y^{\prime}}{\tan \alpha}$
The variables in this equation are defined in Table 1. Equation 1 can be applied to off-ramp junctions with curved or


FIGURE 3 Initiate steering-control Point $P$.
diamond-type exit ramps. Distance $y^{\prime}$ is the area at the beginning of the exit ramp used by the exiting driver as the reference for the initial detection of the exit ramp motion. This distance must be long enough to provide adequate distance for the exiting maneuver to be made smoothly. Depending on ramp design, this reference point may vary considerably. Fortu-
nately, as is obvious from Equation 1, the detection distance is not sensitive to $y^{\prime}$ over any reasonable range.

## Steering-Control Zone Length

Given the point at which steering from the freeway begins, the distance required for the driver to complete the steering maneuver can be estimated. The work of McRuer et al. (5), which provides a completion time for normal drivers, was used. Knowing the freeway speed, the length required for a driver to complete the steering maneuver ending on the SCL can be calculated as
$L_{\mathrm{SC}}=V_{d} \times \mathrm{SC}_{t}$
where $V_{d}$ is the vehicle divergence speed and $\mathrm{SC}_{t}$ is the steering-control maneuver time.

However, when the steering-control maneuver is complete, as shown by Point $T$ in Figure 4, the driver approaching the exit ramp will probably require distance to reorient to the

TABLE 1 GLOSSARY

|  | , |  |
| :---: | :---: | :---: |
| \|Variable| | Definition | Units |
|  |  |  |
| a | Lateral distance or effective moving visual width |  |
|  | of a vehicle, $a \approx 6$. | feet |
| $\alpha$ | Angle of divergence, for diamond type exit ramps, |  |
|  | $2^{\circ} \leq \alpha \leq 5^{\circ}$ (2). For curved exit ramps, equivalent |  |
|  | divergence angle $2^{\circ} \leq \alpha \leq 3^{\circ}$ (2). | degrees |
| $1 \mathrm{~d}_{\mathrm{Bd}}$ | Instantaneous braking deceleration for diamond ramp. | $\mathrm{ft} / \mathrm{s} / \mathrm{s}$ \| |
| $\mathrm{d}_{\mathrm{G}}$ | Coasting deceleration, $\mathrm{d}_{\mathrm{G}} \approx 2$. | $\mathrm{ft} / \mathrm{s} / \mathrm{s}$ |
| e+f | Superelevation plus coefficient of friction, $\mathrm{e}+\mathrm{f} \approx 0.2 .1$ |  |
| $\mid h_{1}$ | Lateral distance from a midpoint between the driver's\| |  |
|  | eyes to a point situated on the right edge of the |  |
|  | rightmost freeway lane, $\mathrm{h}_{1}=\left(\mathrm{W}_{\mathrm{f}} / 2\right)+\mathrm{W}_{\mathrm{DV}}$. | feet |
| \| $\mathrm{h}_{2}$ | Lateral distance from a point on the left edge of |  |
|  | the deceleration lane to the midpoint between the |  |
| 1 \| | driver's eyes in feet, $\mathrm{h}_{2}=\left(W_{D} / 2\right)-W_{D V}$. | feet |
| $\mathrm{L}_{B}$ | Braking distance. | feet |
| $1 L_{B C}$ | Braking distance for curved off-ramps. | feet |
| $L_{\text {Bd }}$ | Braking distance for diamond off-ramps. | feet |
| $L_{\text {Detl }}$ | Distance from wedge point where driver initiates |  |
|  | steering control response. | feet |
| $\mathrm{I}_{\text {Det2 }}$ | Distance from wedge point where driver begins |  |
| 1 \| | transition to ramp tracking. | feet |
| $\mathrm{I}_{\mathrm{G}}$ | Deceleration-in-gear distance. | feet |
| $L_{\text {Lc }}$ | Deceleration-in-gear distance for curved off-ramps. | feet |
| $L_{\text {Gd }}$ | Deceleration-in-gear distance for diamond off-ramps. | feet |
| $L_{\text {Sc }}$ | Steering control distance, $L_{S C}=V_{d} * S C_{t}$. | feet |
| Iscmax | Maximum steering control distance, $I_{\text {scmax }}=L_{\text {Det }}-I_{\text {Det } 2} \mid$ | feet |
| R | Minimum radius of curvature, $\mathrm{R}=\mathrm{V}_{\boldsymbol{C}} /[15(\mathrm{e}+\mathrm{f})]$ (2) | feet |
|  |  |  |

TABLE 1 (continued on next page)

TABLE 1 (continued)

| , |  |  |
| :---: | :---: | :---: |
| \|Variable| | Definition | Units |
|  |  |  |
| S | Longitudinal sight distance to a point on the inside |  |
| \| | aurve. | feet |
| $s c_{t}$ | Steering control maneuver time, $\mathrm{SC}_{t}=1.5$ (5) . | seconds |
| SCL | Speed change lane. |  |
| SCLU | Speed change lane length. | feet |
| $\mathrm{V}_{\mathrm{c}}$ | Controlling speed on curve. | mph |
| $\mathrm{V}_{\mathrm{d}}$ | Vehicular divergence velocity, $\mathrm{V}_{\mathrm{d}} \approx \mathrm{V}_{\mathrm{f}}$. | $\mathrm{ft} / \mathrm{s}$ |
| $\mathrm{V}_{\mathrm{f}}$ | Vehicular freeway velocity. | $\mathrm{ft} / \mathrm{s}$ |
| $\mathrm{V}_{\mathrm{G}}$ | Vehicular coasting velocity. | $\mathrm{ft} / \mathrm{s}$ |
| $\mathrm{V}_{\text {Gfc }}$ | Velocity of the driver at the end of the |  |
|  | deceleration-in-gear phase. For short $L_{G}, \mathrm{~V}_{\mathrm{GfC}} \sim \mathrm{V}_{\mathrm{d}}$. |  |
| 1 | Exact equation: $\mathrm{V}_{\mathrm{GfC}}=\operatorname{SQRT}\left(\mathrm{V}_{\mathrm{d}}{ }^{2}-2\left(\mathrm{I}_{\mathrm{GC}}\right) \mathrm{d}_{\mathrm{G}}\right)$. | $\mathrm{ft} / \mathrm{s}$ |
| $\mathrm{V}_{\mathrm{Gfd}}$ | Velocity of the driver at the end of the |  |
| \| | deceleration-in-gear length. | $\mathrm{ft} / \mathrm{s}$ |
| $\mid \omega_{b}$ | Braking angular velocity, $\omega_{\mathrm{b}}=\mathrm{V}_{\mathrm{Gfc}}{ }^{\mathrm{K}} / \mathrm{/}\left(\mathrm{~S}^{2}+\mathrm{y}^{2}\right)$. | rads/s |
| $\mathrm{W}_{\mathrm{D}}$ | Width of deceleration lane, $\mathrm{W}_{\mathrm{D}}=12$. | feet |
| $\mathrm{W}_{\mathrm{DV}}$ | Lateral distance from the driver's eyes to the |  |
| 1 | longitudinal centerline of the vehicle, $\mathrm{W}_{\text {DV }} \approx 1.5$ | feet |
| $\\| W_{f}$ | Width of an Interstate freeway lane, $\mathrm{W}_{\mathrm{f}}=12$. | feet |
| $\mid \omega_{t}$ | Angular velocity threshold, $\omega_{t} \sim 0.004$ (4). | rads/s |
| $1 \mathrm{w}_{\mathrm{v}}$ | Width of a passenger car, $\mathrm{W}_{W}=7$ (2). | feet |
| $\\|^{\prime}$ | Lateral distance from a point located on the right |  |
| 1 \| | edge of the rightmost freeway lane to a point on the |  |
| 1 | left edge of the exit ramp in feet, $y^{\prime} \approx 8$. | feet |
|  |  |  |

environment that will now be in rectilinear motion relative to the SCL. Point $T$ also represents the location at which the driver may be expected to begin deceleration in gear. The end points of the $L_{\mathrm{SC}}$ segment are Points $P$ and $T$; these points represent the place the driver initiates the stecring-control maneuver and the point at which the maneuver is complete, as shown in Figures 3 and 4.


FIGURE 4 Complete steering-control Point $T$.

## Confirmation of the Appropriateness of the First Detection Distance and the Steering-Control Length

Point $Q$ in Figure 4 is the place at which the driver detects the lateral motion of the point of focus while on the SCL lane (not on the freeway). It is hypothesized that when the point of focus on the left edge of the exit ramp near the wedge point attains an angular velocity approximately equal to 0.004 $\mathrm{rad} / \mathrm{sec}$ (4) anci vehicle velocity and lateral position are known, the longitudinal location of Point $Q$ can be derived. If coasting distance is provided after Point $Q$, Point $Q$ will demarcate that portion of the coasting length after which the driver will anticipate a control response. The longitudinal distance between Point $Q$ and the wedge point is labeled $L_{\mathrm{Det} 2}$. The second detection distance is derived from the following equation:
$L_{\text {Det2 }}=\left[\frac{V_{d}}{\omega_{z}}\left(y^{\prime}-h_{2}\right)-\left(y^{\prime}-h_{2}\right)^{2}\right]^{1 / 2}-\frac{y^{\prime}}{\tan \alpha}$

Equation 3 may also be used for off-ramp junctions with either curved or diamond ramp types. The same precautions and
considerations taken in the application of Equation 1 are to be used with Equation 3.
Once the positions of Points $P$ and $Q$ are obtained with respect to the wedge point, the maximum length of the steeringcontrol zone is derived by subtracting the longitudinal distance between Point $Q$ and the wedge point $L_{\text {Detz }}$ from the longitudinal distance between Point $P$ and the wedge point $L_{\text {Det }}$. This maximum steering-control distance is the distance between the point at which the driver detects the initial motion of the off-ramp from the rightmost freeway lane and the place where the driver detects the initial motion of the offramp from the SCL. The equation for the maximum steeringcontrol length is as follows:
$L_{\mathrm{SC}_{\text {max }}}=L_{\text {Det1 }}=L_{\text {Det2 }}$
Equations 3 and 4 are not essential to the model and are presented only to confirm that $L_{\text {Det }}$ and $L_{\mathrm{SC}}$ occur before the point (Point $Q$ ) at which the driver detects the initial motion of the off-ramp while on the SCL.

## Braking Distance in Response to Ramp Curvature

The longitudinal location of Point $R$ from the controlling point depends on whether the exit ramp is curved (e.g., ramps in cloverleaf interchanges) or straight (e.g., ramps in diamond interchanges), as shown in Figure 5. For the curved exit ramp, Point $R$ represents the beginning of a transitional period in the driver's navigation of the ramp; the driver is switching from compensatory tracking of the SCL to pursuit tracking of the curve. Approaching the curve, the driver initially scans points on the inner edge of the curve at the forward reference distance ( $S$ ). This sight distance is a function of the driver's specd and lateral distance of the lane edge (6). When this focal point on the inner edge of the curve reaches an angular velocity in excess of $0.1 \mathrm{rad} / \mathrm{sec}$, it is hypothesized that the driver will begin braking as part of the transition to pursuit tracking of the ramp curvature. This point, called the driver's braking angular velocity threshold $\omega_{b}$, occurs at distance $L_{B c}$ from the wedge point. The equation for deriving $L_{B c}$ is as follows:

$$
\begin{align*}
& L_{B_{c}}= S \\
&-\left\{R^{2}-\left[\left(\frac{-V_{G_{k}}^{2}}{4 \omega_{b}^{2}}-S^{2}\right)^{1 / 2}\right.\right.  \tag{5}\\
&\left.\left.-\frac{V_{G_{f}}}{2 \omega_{b}}+R+W_{D}-h_{2}\right]\right\}^{1 / 2}
\end{align*}
$$

In summary, $L_{B_{c}}$ is the distance from Point $R$ to the wedge point, as shown in Figure 5. Point $R$ denotes the driver's position when the point of focus on the inner edge of the curved exit ramp at a certain sight distance attains the driver's braking angular velocity threshold; Point $R$ is where the driver will initiate braking with respect to the controlling point, Point $S$.

## Braking in Response to a Ramp Terminus

In the case of diamond exit ramps, the diamond junction control point is usually near the ramp terminus (e.g., traffic


FIGURE 5 Begin-braking deceleration Point $\boldsymbol{R}$.
signal, stop sign, or end of a queue of vehicles). In this case, braking deceleration is hypothesized to begin at Point $R$ when critical elements at the ramp terminus, such as Point $S$, reach the angular velocity threshold $\left(\omega_{t}\right)$ as shown in Figure 5. The distance $L_{B_{d}}$, required to stop at the junction point on a diamond ramp can be calculated from the following equation:
$L_{B_{d}}=\left(\frac{V_{G_{l d}} a}{\omega_{t}}-a^{2}\right)^{1 / 2}$
$L_{B d}$ is the distance between Points $R$ and $S$.
It is further hypothesized that the driver will decelerate at a rate sufficient to maintain the angular velocity of the critical elements at threshold. This braking deceleration $d_{B_{d}}$ is essentially a function of the speed of the driver at the end of the deceleration-in-gear phase $V_{G_{j d}}$ and length $L_{B d}$. The following equation computes the instantaneous braking deceleration required to satisfy this criterion:
$d_{D_{d}}=\frac{\omega_{1} V_{G_{f i}}}{2 a} L_{B_{d}}$

## Coasting Deceleration Until Ramp Angular Velocity Enters Action Field

Once the appropriate $L_{\mathrm{SC}}$ and $L_{B}$ distances have been determined, the length of the SCL is known, assuming that no coasting length is necessary. The coasting segment is the distance that is not required for steering control and braking; the segment provides time for the driver to reorient to rectilinear motion after completing the steering maneuver and to anticipate the required steering-control and braking response for the exit ramp. For curved and diamond exit ramps, the equation for the distances needed for deceleration in gear, $L_{G_{c}}$ and $L_{G_{d}}$, is as follows:
$L_{G_{c}}=L_{G_{d}}=50$ to 100 ft

## Total SCLL

After the three component distances of the SCL have been individually determined for a curved or diamond-type exit ramp, the total length of the SCLL is known. For off-ramp
junctions with curved exit ramps,
$\mathrm{SCLL}=L_{\mathrm{SC}}+L_{G_{c}}+L_{B_{c}}$
For ramp junctions with diamond-type exit ramps,
$\mathrm{SCLL}=L_{\mathrm{SC}}+L_{G_{d}}+L_{B_{d}}$
The control point, Point $S$, is determined by the highway design engineer on the basis of such factors as the amount of right-of-way available, interchange type, radius of curvature, divergence angle, exit ramp volume, and ramp vehicle storage. Curved exit ramps could be designed without a controlling curve, so that the control point becomes the ramp terminus. In such a case, no braking zone would be given near the freeway; drivers would coast most of the way on the ramp and then brake near the terminus. Current practice is to have the driver brake twice, once just before the curve and the second time near the terminus. Diamond-type exit ramps should be designed so that their lengths place the braking that occurs near the terminus far enough away from the freeway.

## TEST OF THE EXIT MODEL

## Procedure

The driver behavioral model of diverging was evaluated from data collected in an NCHRP study (Reilly et al., unpublished data). The evaluation of the model tests the model's ability
to estimate (a) the distance $L_{\text {Det1 }}$, from the wedge point at which the driver initiates a steering-control maneuver; (b) the length $L_{\mathrm{SC}}$, of the steering-control distance; and (c) braking distance $L_{B}$. To determine the actual steering-control distances in the field, the divergence speed of the vehicle $V_{d}$ and the time $t_{\mathrm{SC}}$ taken to complete the steering-control maneuver were measured. The braking distance derived from the data was calculated from the average speed during braking $V_{b}$ and the duration of the braking $t_{b}$. Once these three distances were determined, they were compared to the distances produced by the model.

## Sites

The seven off-ramp sites used in the confirmation of the model are listed in Table 2. Three sites are in Arizona, three in California, and one in Illinois. Five of the seven sites have diamond-type ramps, and the remaining two have curved exit ramps. Six sites have a taper-type SCL, and one has a parallel-type SCL.

## Data Reduction and Analysis

Typically, data were collected at the various sites using one videocamera mounted on an overpass directly overlooking the off-ramp junction. In some rare cases (i.e., unique site characteristics), two video cameras were used, one placed on top of a vehicle to achieve the optimum camera angle. The

TABLE 2 EXIT RAMP SITES

| Site | Location ${ }^{\text {a }}$ | Route ${ }^{\text {a }}$ | Rarmp Type ${ }^{\text {a }}$ | SCL Type ${ }^{\text {a }}$ |
| :---: | :---: | :---: | :---: | :---: |
| 1. | NB Ajo Way Exit <br> Tucson, Arizona | I-19 | Diamond | Taper |
| 2. | NB Atlantic Blvd. Exit Los Angeles, California | I-5 | Diamond | Taper |
| 3. | EB Hotel Circle Exit San Diego, California | I-8 | Curved | Taper |
| 4. | NB Quince St. Exit San Diego, California | SR-163 | Diamond | Taper |
| 5. | EB Rural Rd. Exit Phoenix, Arizona | SR-360 | Diamond | Taper |
| 6. | EB Alma School Rd. Exit Phoenix, Arizona | SR-360 | Diamond | Taper |
| 7. | NB Deerfield Rd. Exit Chicago, Illinois | $\mathrm{RT}-41$ <br> Edens | Curved | Parallel |

$\mathrm{a}_{\text {Sounce: }}$ Reilly et al., NCHRP 3-35, unpublished data.
right shoulder and the rightmost edge of the exit ramp were delineated by fluorescent orange traffic cones placed at 50 or $100-\mathrm{ft}$ intervals, depending on the length of the site and the angle of divergence or the curvature. The cones extended from a point in front of the beginning of the taper to the point on the exit ramp where the controlling curve could first be seen. For diamond-type off-ramps, the cones were extended down the exit ramp as far as possible given the range of the camera lens.

The data were processed by viewing the videotapes and logging the time displayed on the tape when the front or rear tires, depending on the camera orientation, of selected exiting vehicles reached each cone along the length under observation. Also noted in the processing of the data were the type of vehicle exiting the freeway (i.e., passenger car, truck, bus, or recreational vehicle), the point at which the driver initiated the steering-control maneuver, the point at which the exiting driver completed the steering-control maneuver, and the position of the vehicle at which the driver initiated braking. Braking was observed on nearly all exit ramps; at certain ramps in diamond interchanges, the camera could not track the exiting vehicle to the ramp terminus.

These data were coded into a microcomputer spreadsheet program and analyzed. Because the time at which the vehicle reached a cone and the trap length (distance between two cones) were known, vehicular speed and acceleration profiles along the distance travelled were computed and reproduced graphically.

## RESULTS

The driver behavioral model of diverging was validated by examining three different aspects: (a) its estimation of the distance from the wedge point at which exiting drivers initiate a steering-control response, (b) its ability to set the boundaries of the segment where drivers perform their steering-control maneuvers, and (c) its potential for estimating the places where drivers initiate a braking response to safely navigate critical curves for curved exit ramps or stop at ramp termini for diamond exit ramps.

The $L_{\text {Den }}$ distances from field data and from the model are presented in Table 3. The average difference between model
and field ranged from -57 to 86 ft . A hypothesis test using the Student $t$-statistic was performed. At the 5 percent level of significance, the hypothesis that there is no difference between average model $L_{\text {Det }}$ value and field $L_{\text {Det }}$ value could not be rejected. A 5 percent level of significance was selected for use throughout this study because of the nature of the traffic data and because the resulting confidence intervals were acceptable.
The estimated and observed steering-control distances were compared. The results are presented in Table 4. A Student $t$-test of the differences between the predicted and observed distances indicated no significant differences at the 5 percent level. Furthermore, the $L_{\mathrm{SC}_{\text {max }}}$ value (for $\omega_{t}=0.004 \mathrm{rad} / \mathrm{sec}$ ) was greater than the 95 percent confidence interval of measured $L_{\mathrm{Sc}}$ in all cases. The results confirmed previous research (5).

Applying Equation 5 resulted in estimated braking distances before the wedge point for curved exit ramps. Input variables $\omega_{b}, S, h_{2}$, and $W_{D}$ in Equation 5 were assigned their default values. Variable $V_{G / c}$ was estimated by setting it equal to the average deceleration velocity $V_{G}$ of Site 3 or by reading it directly from the velocity-distance profile of Site 7. Variable $R$ was approximated by setting it equal to the average braking velocity $V_{b}$ of Site 3 or by setting it equal to the minimum speed observed on the exit ramp in the velocity-distance profile of Site 7. The results of the comparison between the observed and predicted braking distances from the beginning of the curve are presented in Table 5. The lack of variability in the field data allowed no statistical tests of the significance of the observed differences. However, the predicted values were consistent with the observed values.

Diamond-type exit ramp data were collected at two sites. Use of Equations 6 and 7 produced an approximation of the braking distances required for the driver to stop at the critical point. The critical point was assumed to be the rear end of a passenger car. In Equation 6, the input variables $\omega_{t}$ and $a$ were set at $0.004 \mathrm{rad} / \mathrm{sec}$ and 6 ft , respectively. Variable $V_{G / d}$ was set equal to the average braking speed measured at Sites 2 and 4. After braking distance $L_{B_{d}}$ was calculated from Equation 6, it was input into Equation 7 to derive $d_{B d}$. The model's braking distances and their corresponding field-derived braking distances were compared, and the results are presented in Table 6. The differences between observed and

TABLE 3 BEGIN-STEERING-CONTROL POINT FROM WEDGE POINT VALIDATION

| Site | n | Observed <br> (Average) | Predicted | Difference <br> (feet) |
| :--- | ---: | :---: | :---: | :---: |
| 1. | 47 | 300 | 310 | 10 |
| 4. | 65 | 211 | 297 | 86 |
| 5. | 65 | 268 | 325 | 57 |
| 6. | 124 | 398 | 341 | -57 |
| 7. | 11 | 350 | 293 | -57 |

TABLE 4 STEERING-CONTROL DISTANCE VALIDATION

| Site | n | Observed <br> (Average) | Predicted | Difference <br> (feet) |
| :--- | :---: | :---: | :---: | :---: |
| 1. | 47 | 98 | 113 | 15 |
| 2. | 30 | 58 | 87 | 29 |
| 3. | 54 | 58 | 80 | 22 |
| 4. | 65 | 79 | 107 | 28 |
| 5. | 65 | 135 | 119 | -16 |
| 6. | 124 | 202 | 126 | -76 |
| 7. | 11 | 175 | 106 | -69 |
| Average difference $=-9 \%$ |  |  |  |  |

TABLE 5 COMPARISON BETWEEN OBSERVED AND PREDICTED BRAKING DISTANCES FOR CURVED RAMPS

| Site | n | Observed <br> (Average) | Predicted | Difference <br> (feet) |
| :---: | :---: | :---: | :---: | :---: |
| 3 | 54 | 100 | 64 | -7 |
| 7 | 11 | 25 | 0 | -15 |

TABLE 6 COMPARISON BETWEEN OBSERVED AND PREDICTED BRAKING DISTANCES FOR DIAMOND RAMPS

| Site | n | Observed <br> (Average) <br> (feet) | Predicted <br> (feet) |
| :---: | :---: | :---: | :---: |
| $\omega_{\mathrm{t}}=0.004$ radians/second |  |  |  |$|$| 253 |  |  |
| :---: | :---: | :---: |
| 4 | 65 | 169 |

predicted distances were significant at the 5 percent level. Thus, the braking distance model could not be called a correct representation of driver behavior on diamond ramps.

## DISCUSSION OF RESULTS

## General Remarks and Assumptions

The significant differences between the observed and predicted begin-braking distances for the diamond ramp were surprising. The basic model for this component derives directly from standard car-following theory, which has been extensively validated by empirical methods. The model is a simple closed-loop one based on the principle that the driver maintains the angular velocity of the overtaken vehicle at threshold. The speed profile and, hence, the instantaneous
deceleration, can be directly derived by means of Equation 7.

The begin-braking distance had to be derived from the time and distance plots. The speed was calculated by estimating the time spent in each trap. Thus, speed was simply the time spent in a known length of road. Any changes in speed from trap to trap were used to estimate acceleration. With the acceleration and knowing the terminus of the ramp, an estimate of where the deceleration began was derived as a basis of testing the model. However, given that the estimated speed at the beginning of this deceleration period was derived from the average speed from the coasting zone, the estimate is unreliable. Similarly, acceleration derived from video recordings is notoriously unreliable. The empirical data are far too unreliable to be a direct test of the model. A direct test of the model would require simply a measure of the distance at which drivers initiate deceleration. Practically, all that is
needed is a measure of the distance from the ramp terminus to the position where brake lights first occur. If the instantaneous deceleration could be measured over this distance, an even more complete test of the model would be possible. Passive measurements of traffic, such as by video recording, make this practically impossible.

With the exception of the braking distance for diamond offramps, the proposed model provides a consistent description of the diverge process used by drivers. The validation is reasonable but not robust. One of the problems of testing a rather complex model such as that proposed in a free-field environment is the lack of control over the variables critical to the model. For example, the model is essentially mechanistic and takes no account of higher-order cognitive behavior. In the test sites used, most of the drivers observed were probably regular users. These drivers have learned both the geometric properties of the interchange and the traffic. Thus, they have a set of overlearned response tendencies. This tendency should lead to major modifications in diverging behavior.

Also, neither of the curved ramps had constant radii; they were spirals. In this analysis, an equivalent constant radius was used to approximate the curve. This situation probably explains why the begin-braking actions occurred at angular velocities below $0.1 \mathrm{rad} / \mathrm{sec}$.

In the figures, the model assumed a parallel-type SCL. However, a taper-type SCL can also be assumed, provided that the taper width when the driver is at Point $P$ is 12 ft . Moreover, any deceleration lane length provided before Point $P$ is not necessary. Savings in pavement costs could be obtained if future SCLs do not have pavement before Point $P$.

An implicit assumption of the model is that the driver carries out one decision-and-control operation at a time throughout the exiting process. This assumption is a fairly standard interpretation of optimum human guidance and control. Highways designed so that the driver can sequence the tasks and thereby minimize control errors are recommended. Rapid alternation among tasks (e.g., braking and steering), not uncommon in highway system operation, may be considered a symptom of poor design.

Another assumption is that the fluorescent cones used in the data collection process did not contaminate driver behavior. The cones were placed as far away from the lane of travel as possible to prevent the drivers from using them as guidance markers. This process was especially needed for the initial detection distance, where the cones were well beyond the decision point and off the far shoulder of the SCL.

Finally, the model assumed that each vehicle measured was independent, and no other vehicles were on the SCL at the same time. In most cases, especially in peak hours, this is rarely the case. When a number of vehicles are on the SCL and exit ramp, the driver must respond to the presence of the vehicles ahead as well as to the genmetry. However, in the data used in the analysis, the vehicles were independent.

Given these limitations, the model appears to be a reasonable representation of the diverge and exiting process. The model is most applicable to drivers in unfamiliar environments in low-to-moderate ramp volume situations. In this sense, the model is conservative. The model probably defines the longest SCL requirements for normal passenger car driving. Although the model was developed assuming an SCL and ramp without grade, it can easily be adapted to explain the response to
those conditions. In general, the model provides a rational means for the design of freeway exits. A step-by-step procedure of the model's application to geometric exit design is given in the following section.

## Coasting Distance

Historically, rates of deceleration have been observed on SCLs, and their coasting phase has been built into SCL design standards (2). However, coasting is hypothesized to reflect driver reorientation to compensatory tracking after completion of the steering maneuver and to the angular velocity of the geometric elements of the exit ramp. At the end of the steeringcontrol phase, the angular velocity of the ramp elements have decreased significantly and have changed relative to the driver. The motion detection of the exit ramp for a driver on the SCL may serve as an anticipatory cue for a speed or steering control change for entry onto the exit ramp. One response on the SCL is to coast, that is, to decelerate the vehicle while in gear.

AASHTO policy is to provide a length of ramp for coasting, so that the exiting vehicle can decelerate to exit ramp speed (2). The model provides a coasting distance so that the driver has time to recover from the steering maneuver onto the SCL and anticipate an appropriate response to the exit ramp geometry. The underlying issue is really one of the time and distance the driver may require to move from the steering control to compensatory tracking of the SCL, before any control response required by the ramp. Obviously, the length of the SCL must be at least as long as $L_{\mathrm{SC}}$, or the two tasks overlap. In this case, the driver will probably decelerate on the freeway rather than on the SCL. On one curved exit ramp with an SCL shorter than $L_{\mathrm{SC}}$, exiting drivers used the freeway lane to decelerate. In essence, they used sufficient distance to move from compensatory to pursuit tracking of the ramp curve.

The length of the SCL after completion of the steeringcontrol maneuver appears to depend on three factors. One is the variability of the angular velocity threshold and the steering-control length. Angular velocity threshold varies by driver. Moreover, the angular velocity threshold has a log normal distribution with a median value of $0.004 \mathrm{rad} / \mathrm{sec}$ (4). If $\omega_{\mathrm{r}}<0.004 \mathrm{rad} / \mathrm{sec}$, drivers will diverge later than predicted; if $\omega_{t}>0.004 \mathrm{rad} / \mathrm{sec}$, drivers will diverge earlier. Steeringcontrol maneuver time also varies by driver. If $\mathrm{SC}_{4}>1.5$ sec, $L_{\mathrm{SC}}$ will be greater than the model's predicted value; if $\mathrm{SC}_{r}<1.5 \mathrm{sec}, L_{\mathrm{SC}}$ will be less, assuming constant divergence speed in both cases.

The second factor is the time required by the driver to reorient visually to the critical elements of the ramp diverge area after completing the steering control. The driver must adjust to tracking the SCL and attend to the ramp curvature. This adjustment should not require more than 0.5 to 1.0 sec or, normally, 50 to 100 ft .

The third factor is the ramp curvature, or the effective ramp terminus in the case of the tangent off-ramp. The angular velocity of the curve increases nonlinearly as the distance to the beginning of the curve decreases. This action is a function of speed and radius of curvature. The srinaller the radius, the greater will be the distance at which angular velocity will reach the pursuit-tracking magnitude. This distance should deter-
mine the begin-braking point. Ideally, the SCLL should be long enough to minimize the deceleration required for the driver to negotiate the curve (i.e., an angular velocity below 0.1 to $0.3 \mathrm{rad} / \mathrm{sec}$ ). However, if the SCL is long enough to meet the diverge criterion, it will always be long enough to meet the curve-tracking criterion. The model clearly suggests that the distance at which divergence from the freeway begins is the critical determinant of SCLL.

## Procedure for Determining SCLL for Curved OffRamps

To determine the appropriate length of an SCL that has an exit ramp of constant curvature, the following procedure must be performed:

1. Determine the distance from the wedge point at which the driver, while in the rightmost freeway lane, initiates a steering maneuver onto the SCL (i.e., apply Equation 1).
2. Calculate $L_{\mathrm{SC}}$ by using Equation 2.
3. Derive the distance from the wedge point to the place where the driver initiates braking by using Equation 5. To solve Equation 5, perform the following steps:
-Determine $R$. If $R$ is not known, assume the controlling speed of the curve, $R=V_{c}^{2} /[15 *(e+f)]$.
-Estimate $V_{G_{f c}}$ for minimum coasting distance, $V_{G_{f c}} \approx$ $V_{d}$.
-Estimate $S$ using the $V_{G f_{c}}$ value from previous step, $S$ $=3.41 * V_{G f c}$, or consult Figure $6(6)$.


FIGURE 6 RD reference point within visual field.
-Use Equation 5 to obtain a value for $L_{B_{c}}$.
If $L_{\text {Det2 }}<L_{B c}$, a spiral curve, rather than a curve of constant radius, should probably be used. If the exact values for $V_{G_{f}}, S$, and $R$ are known, Table 7 can be used instead of Equation 5.
4. Compute the coasting distance by using Equation 8.
5. Determine the SCLL from Equation 9.

## Procedure for Determining SCLL for Tangent OffRamps

In order to calculate the SCLL required for diamond-type exit ramps, the following procedure must be completed:

TABLE 7 BRAKING DISTANCES FOR CONSTANT-RADIUS EXIT RAMPS

| $\begin{gathered} \mathrm{V}_{\text {Gfc }} \\ (\mathrm{mph}) \end{gathered}$ | $\begin{array}{r} S \\ \text { (feet) } \end{array}$ | 15 75 | 20 133 | 25 208 | $\begin{array}{r} 30 \\ 300 \end{array}$ | 35 408 | $\begin{array}{r} 45 \\ 675 \end{array}$ | $\begin{aligned} & V_{C}(\text { mph }) \\ & R(\text { feet }) \end{aligned}$ |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: |
| 30 | 150 | $79^{\text {a }}$ | 45 | 13 | 0 | b - | - |  |
| 35 | 175 | 101 | 63 | 27 | 0 | 0 | ${ }^{\mathrm{b}}$ - |  |
| 40 | 200 | 125 | 82 | 43 | 6 | 0 | - |  |
| 45 | 225 | ${ }^{\text {c }}$ | 102 | 60 | 20 | 0 | 0 |  |
| 50 | 250 |  | 124 | 78 | 35 | 0 | 0 |  |
| 55 | 275 |  | 146 | 97 | 51 | 6 | 0 |  |
| 60 | 300 |  | 169 | 116 | 67 | 21 | 0 |  |
| 65 | 325 |  | 192 | 136 | 85 | 35 | 0 |  |
| 70 | 350 |  | 217 | 157 | 103 | 51 | 0 |  |

$$
\text { a } \begin{aligned}
a_{\text {Note }} \text { foveal } & \begin{aligned}
\text { field } & =3 \text { degrees } \\
\omega_{\mathrm{b}} & =0.1 \mathrm{rad} / \mathrm{sec} \\
\mathrm{e}+\mathrm{f} & =0.2 \\
\mathrm{~h}_{2} & =4.5 \text { feet } \\
\mathrm{W}_{\mathrm{D}} & =12 \text { feet }
\end{aligned} \text { 据 }
\end{aligned}
$$

$\mathrm{b}_{\text {No braking occurs before the wedge point and after Point } Q \text {. If }}^{\text {braking is initiated, then it must happen on the exit ramp. }}$
$\mathrm{C}_{\mathrm{y}}>\mathrm{R}+\mathrm{W}_{\mathrm{D}}-\mathrm{h}_{2}$

1. Perform Steps 1 and 2 as described for the curved off-ramp.
2. Calculate the required braking distance using Equation 6. Braking distances are also shown in Table 8, assuming default values.
3. Derive the necessary deceleration-in-gear distance by means of Equation 8 after assuming an average coasting deceleration $d_{G}$.
4. Determine the SCLL using Equation 10.

## SCLL Needed for Curved Off-Ramps of Various Types

Many curved exit ramps do not have curves of constant radii for long ramps. These curves with varying radii are transitory curves; that is, they begin at the tangent part of the SCL (infinite radius), then change to a spiral curve (varying from infinite radius to the minimum radius), and then to a constant curve at the minimum radius $R_{\min }$, for a short distance as shown in Figure 7. In these curves of varying radii, the coasting length extends past the wedge point and onto the exit ramp. If $R_{\min }$ is great enough, the coasting length will end the way it ends at a diamond-type exit ramp, because braking will


FIGURE 7 Layout of a spiral-curve exit ramp.
not be required in advance of $R_{\text {min }}$. If $R_{\min }$ is small, the driver will brake before entering the $R_{\text {min }}$ arc. This initial braking point puts the driver somewhere past the wedge point and on the spiral curve. In such a case, the driver is no longer in rectilinear motion. When the driver is in curvilinear motion, a different set of equations must be considered in determining at what distance the driver's point of focus reaches the braking angular velocity threshold. Briefly, for the model to handle exit ramps with varying radii, Equation 5 must be modified to reflect the curvilinear motion, and an equation to locate points on the inner edge of the curve in terms of the $x, y$ coordinate system must also be incorporated.

The highway design engineer should consider the use of a transition curve only after applying the model for constant curve ramps. If $L_{D e t 2}<L_{B c}$, a transitory curve must be considered, because the SCL is not providing enough length.

## SUMMARY AND CONCLUSION

The behavioral model of diverging presents a different way of defining the deceleration length required in the design of freeway SCLs in off-ramp junctions. Such a model offers a rational basis for design rather than the usual empirical means based on vehicle characteristics. The human factors model provides insights into traffic operations in off-ramp junctions beyond those derived from aggregate analysis of vehicle behavior. The results of the model's validation tests using video recordings of traffic on seven off-ramp sites were consistent with the model's estimates of the initial diverge distance $L_{\text {Det } 1}$, the steering-control length $L_{\mathrm{SC}}$, and the braking distance $L_{B_{c}}$ for curved off-ramps.

Five sites were used in the validation of $L_{\mathrm{Det},}$. Seven sites were used to determine the longitudinal distance drivers needed to perform their steering-control maneuver onto the SCL. The comparison between the model's $L_{\mathrm{SC}_{\text {max }}}$, assuming that $\omega_{t}=$ $0.004 \mathrm{rad} / \mathrm{sec}$, and the measured $L_{\mathrm{SC}}$ revealed that $L_{\mathrm{SC}_{\text {max }}}$ was greater than $L_{\mathrm{SC}}$ in all cases. The $L_{\mathrm{SC}_{\text {max }}}$ value is useful in preventing the overdesigning the SCLL. If $L_{S C}$ is made greater than $L_{\mathrm{sC}_{\text {max }}}$, drivers will use the extra length for acceleration purposes, that is, use the deceleration lane as a freeway lane.

TABLE 8 BRAKING DECELERATION AND DISTANCE REQUIRED FOR STOPS ON DIAMOND EXIT RAMPS

| Instantaneous measures ${ }^{\text {a }}$ |  |  |  |  |  |  | Average measures ${ }^{\text {b }}$ |  |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: |
| $\underset{(m \underline{G} h)}{V_{\text {Gf }}}$ | $\begin{gathered} \omega_{t}=0 . \\ d_{B d} \\ (\mathrm{ft} / \mathrm{s} / \mathrm{s}) \end{gathered}$ | $\begin{aligned} & 010 \\ & \mathrm{~L}_{\mathrm{Bd}} \\ & \text { (ft) } \end{aligned}$ | $\begin{gathered} \omega_{\mathrm{t}}=0 . \\ d_{\mathrm{Bd}} \\ (\mathrm{ft} / \mathrm{s} / \mathrm{s}) \end{gathered}$ | $004$ $\mathrm{IBd}_{\mathrm{Bd}}$ (ft) | $\begin{gathered} \omega_{\mathrm{t}}=0 . \\ d_{\mathrm{Bd}} \\ (\mathrm{ft} / \mathrm{s} / \mathrm{s}) \end{gathered}$ | $001$ $\mathrm{I}_{\mathrm{Bd}}$ (ft) | Stopping Velocity (mph) | Stopping Deceleration (ft/s/s) |
| 30 | 6.0 | 162 | 3.8 | 257 | 1.9 | 514 | 15 | 1.5 |
| 40 | 9.2 | 188 | 5.8 | 297 | 2.9 | 593 | 20 | 2.4 |
| 50 | 12.8 | 210 | 8.1 | 332 | 4.1 | 663 | 25 | 3.3 |
| 60 | 16.9 | 230 | 10.7 | 363 | 5.3 | 727 | 30 | 4.3 |
| 70 | 21.2 | 248 | 13.4 | 392 | 6.7 | 785 | 35 | 5.4 |

$a_{a}=6$ feet.
$\mathrm{b}_{\mathrm{a}}=6$ feet, $\omega_{\mathrm{t}}=0.004$ radians per second.

The validation tests on the braking lengths for curved and diamond-type exit ramps were carried out using data from two curved and two diamond ramps. For the curved ramps, the field data were consistent with the model estimates; the differences were 7 and 15 ft . For the two diamond ramps, observed lengths were significantly different from the model prediction. These differences appeared to be caused by the inability to determine where braking actually began in the field studies.

A topic of further investigation involves diamond off-ramp termini that do not end at intersections; they terminate at merge or weaving sections on frontage roads and streets. In these cases, the exit ramp drivers do not brake to a stop; rather, they brake to match the speed of the vehicle preceding them or traffic on the frontage road or street.

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# Operational Effects of Larger Trucks on Rural Roadways 

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#### Abstract

Ability of various truck configurations to negotiate rural roads with restrictive geometry was examined in addition to effects of such trucks on traffic operations and safety. Truck sizes included truck-tractor semitrailers with trailer lengths of 40,45 , and 48 ft (i.e., semi-40, semi-45, and semi-48) and twin-trailer combinations with 28 -ft trailers (i.e., twins or double 28 ). Test sites consisted of approximately 60 mi of rural, two-lane roads in New Jersey and California with a variety of lane widths, shoulder widths, and horizontal and vertical alignment. Field testing involved following control trucks of each truck type along the selected routes. Photographic and radar equipment were used in a data collection caravan to measure the effects of the trucks on oncoming vehicles in terms of speed changes and lateral placement changes. Statistical testing was used to compare operational differences between various truck types for specific geometric conditions. Results showed that semi-48 and twins caused some changes in operation of oncoming vehicles, particularly on narrow roadways. However, careful driving by drivers of larger trucks may have partially compensated for operational differences in oncoming vehicles between truck types. Overall, truck driving behavior and site differences had more of an effect on vehicle operations than the effects of the different truck types. Potential safety problems as evidenced by extreme maneuvers were observed for a few oncoming motorists in reaction to the twins and longer tractor semitrailers.


The Surface Transportation Assistance Act of 1982 (1982 STAA) requires that states allow the operation of wider and longer trucks on the Interstate and other designated federalaid highways, termed the National Network. In terms of trailer widths on the National Network, states may not impose width limits more or less than 102 in . (except for Hawaii, which has a 108 -in. maximum). Before 1982, a maximum truck width of 96 in. was commonly used by most states. The 1982 STAA also provided that states allow semitrailers of at least 48 ft operating in a tractor-semitrailer combination and twin $28-\mathrm{ft}$ semitrailers operating with a tractor on the National Network. Many states now allow semitrailers of 53 ft on the National Network.

Serious questions have been raised regarding the safety of these larger trucks and the ability of various portions of the highway system to safely handle such larger trucks. According to STAA, highways on which larger trucks are allowed to operate should be carefully selected to avoid unnecessary hazards to other road users. In order to perform such a selection, the effect of the operation of larger trucks on safety and traffic operations must be evaluated.

[^4]Turnpike and Interstate systems that exist today have generally been built with high geometric standards. However, the federal-aid primary system and secondary system in many instances includes lower geometric designs, which may preclude the safe operation of large trucks. Because of higher speeds on rural highways, potential for severe truck accidents is increased on roads with narrow lanes and shoulders, sharp curves on steep grades, poor sight distance, and hazardous roadside conditions. Existence of such restrictive geometry may impact safety and limit operations of the larger trucks specified in the 1982 STAA. Therefore, the impacts of larger truck operations on restrictive geometry must be evaluated to provide insights relative to appropriate geometric criteria in the truck route selection process for various types of larger trucks.

Ability of various truck configurations to negotiate rural roads with restrictive geometry was determined in addition to effects of such trucks on the traffic operations and safety of such roads. Truck sizes included truck-tractor semitrailers with trailer lengths of 40,45 , and 48 ft and trailer widths of 96 and 102 in . Twin-trailer combinations with 28 -ft trailers were also included. Truck sizes were studied on arterials and collector routes designed to lower standards and not on arterials and freeways.
A 2-year FHWA study dealing with truck effects on rural roads and at urban intersections was used (1). Results dealing with urban intersections were reported previously by Hummer et al. (2). However, operational field testing of various truck sizes at selected rural sites considered to have problem geometrics is discussed. An analysis was conducted of operational field data to determine the effect of larger trucks on the safety and operations relative to oncoming vehicles in the traffic stream. Information also was gained on geometric conditions that may pose a problem for specific truck types.

## BACKGROUND

## Accident Studies

Of the many research studies conducted in recent years on large truck safety and operations, several have compared accident rates of various truck types. A 1988 study by Jovanis et al. (3) found that twins (i.e., tractors with two $28-\mathrm{ft}$ trailers) had a significantly lower accident rate than semis (tractor semitrailers) on Interstate, state, and local roads. The study was based on a matched pair analysis of 3 years of accident and exposure. Stein and Jones (4) found twins to be overinvolved in crashes compared with semis on the basis of data
collected on two Interstate highways in Washington state. Using California data, Graf and Archuleta (5) concluded that twins have higher accident involvement rates than semis on rural roads, but a lower involvement rate on urban streets. On the basis of 5 years of fatal truck accident and exposure data for heavy trucks (greater than $10,000 \mathrm{lb}$ ), Campbell et al. (6) found that twins have a 10 percent higher fatal accident rate nationwide than semitrailers after adjusting for differences in travel by road class, time of day, and area of truck travel.
A 1981 study by Glennon (7) used matched-pair analysis of freight carriers in Pennsylvania (1976 to 1980 data) and found no significant difference in accident rate between twins and semitrailers. Similarly, studies by Chira-Chavala and O'Day (8) and by Yoo et al. (9) also found little or no difference in accident rates between twins and semitrailers. On the basis of a synthesis of prior studies, a 1986 TRB study found twins to be slightly overinvolved in truck crashes compared with semitrailers, but projected a reduction in truck travel from the greater carrying capacity of twins that would offset any accident increase ( 10,11 ).

All of these studies attempted to compare accident rates only between twins and semitrailers, but not between trailer width ( 96 versus 102 in .) or length of trailers (e.g., 45 versus 48 ft ) probably because of the difficulty in obtaining trailer size data for truck accidents and exposure. Several, but not all, of the studies appropriately controlled for highway type, time of day, or driver characteristics that can have a considerable influence on the results.

## Operational Studies

A 1982 field study by Seguin et al. (12) analyzed the effects of truck sizes on certain traffic situations. Methodology involved photographing lateral placement of oncoming vehicles and measuring their speeds from a van following a staged truck that could be expanded to widths of $96,102,108$, and 114 in . Results suggest that vehicles passing any large truck (in the same direction) tend to move away from the center of the lane. Increased widths caused drivers who wished to pass to follow the truck at a greater distance to allow adequate sight distance for passing. However, no increases in shoulder encroachments or acceptances of smaller gaps were found for passers of wider trucks.

A 1981 study by Hanscom (13) included the effects of truck size, configuration, and weight on traffic and trucks for several types of roadway geometrics. Study sites included upgrades, downgrades, curves with grades, a freeway ramp, a freeway merge, and an intersection. Despite numerous operational differences associated with truck size and weight, the observed effects were weak. Typical truck differences found were reduced speeds, higher deviations from traffic mean speeds, and higher clearances of following vehicles, all exhibited by loaded and double-trailer rigs (by comparison with empties and singles, respectively). Negligible operational effects were found to be associated with truck length. Adverse safety effects were most pronounced on upgrades, whereas certain safer behavior was noted for heavier trucks on downgrades. The analysis demonstrated that a maximum of only 37 percent of truck operational effects were explainable by truck size and weight (13).

Numerous other studies have been conducted dealing with a variety of truck safety and operational issues such as truck offtracking (14-16), performance on curves and ramps $(17,18)$, operations of oversized loads (19), critical overturning speeds $(20,21)$, adequacy of current AASHTO design standards to accommodate current truck sizes (22), and others. One of the key unanswered issues still involves the effects of truck sizes and configurations on various rural road situations and types of roads where such trucks should be permitted.

## DATA COLLECTION

## Candidate Study Conditions

Conditions selected included tangent and curve sections of two-lane rural roads, including various roadway widths. Oversized trucks operating on such roads could run off the road or encroach on lanes of high-speed opposing traffic. Numerous combinations of roadway geometry and truck size were examined on the basis of accident potential, traffic flow problems, and available operational parameters to support safety analyses.
Truck types selected include the baseline 40 -ft semitrailer, about which much is known operationally; pre-1982 maximum size 45 - ft semitrailer; post-1982 semitrailer of 48 ft ; and 28 - ft twin-trailer truck. Many 48 -ft semitrailers have rear axles that may be moved forward or backward relative to the cab. Better load-bearing capability is achieved when the axles are back. Because the 48 -ft semitrailer is generally more maneuverable with axles forward, the vehicle was studied with the rear axles positioned forward and backward as far as possible relative to the truck cab.
Measures of effectiveness (MOEs) examined on the rural two-lane roads were all measures of driver behavior while passing trucks from the opposing direction. These MOEs included lateral placement of random oncoming vehicles with respect to the truck's rear tires and changes in lateral placement and speed of opposing vehicles as they approached the truck. Independent variables included various roadway geometric and traffic parameters (tangent or curve, degree of curve, number of curves per mile, speed limit, etc.), opposing vehicle type and size, and truck size.

Use of random oncoming vehicles was considered to provide a representative sample of drivers and vehicles in the traffic stream on the selected routes. This method was used because of the large sample of observations that would probably cancel biases related to driver age, gender, etc. Use of random vehicles in the traffic stream is a technique used by Seguin et al. (12) and Parker (19) in their operational studies of trucks and oversized loads, respectively.

## Data Collection Methods

Collection of the rural two-lane data involved a caravan of three control vehicles traveling along the road encountering free-flow oncoming vehicles, as shown in Figure 1. A lead car was positioned at the head of the caravan. An observer in the lead car informed the other caravan vehicles via radio that a free-flow oncoming vehicle was approaching for study,


FIGURE 1 Overview of rural data collection.
assigned the vehicle an identification number, recorded the oncoming vehicle's speed by use of a moving radar meter, and noted the odomeler reading (for later use in matching MOEs to roadway geometric data). The control truck of known size traveled approximately 0.1 mi behind the lead car. An observer in the truck photographed the oncoming vehicle when directly beside the lead car. A scaled board attached to the rear bumper of the lead car provided a reference so that a slide of the photograph could be used later to measure the oncoming vehicle's lateral placement.

The trail car of the caravan traveled immediately behind the control truck. An observer in the trail car recorded the
speed of the oncoming vehicle while passing the control truck using the moving radar device and another observer photographed the oncoming vehicle when directly beside the rear of the truck. Again, lateral placement of the oncoming vehicle could be determined because a scaled board was also attached to the rear of the truck. Odometer readings were also recorded at that point for verification of the match to roadway geometric data.

Rural two-lanc data were collected over predesignated routes in California and New Jersey for a variety of roadway geometrics, including narrow and wide lanes and shoulders, curves and tangents, and under various traffic volume conditions. Data were collected during daylight hours with dry pavement conditions. The data collection methodology proved accurate and efficient. Speeds were recorded to the nearest mile per hour and lateral placements within approximately 0.2 ft .

## RESULTS

Data collection procedures resulted in the collection of speed and lateral placement data for samples of vehicles opposing each type of control truck (i.e., semi-40, semi-45, semi-48, and double-28) on two-lane rural roads. Each data point was later matched to a detailed set of information on the roadway geometrics at that point. Data analysis mainly involved separating effects of different geometric variables from the effects of truck size so that an accurate assessment could be obtained of differences between truck sizes for various rural conditions.

## MOEs

Information recorded on the operation of each opposing vehicle in the data set included (a) speed at the lead car, (b) speed at the truck, (c) lateral placement at the car (in relation to the centerline and edgeline of the road), and (d) lateral placement at the truck. Other MOEs created from these data included the speed change of opposing vehicle (speed at the lead car minus speed at the truck), lateral placement change of the opposing vehicle (lateral placement at the lead car minus the lateral placement at truck), and clearance between the truck and opposing vehicle. Several discrete (yes or no) variables were also analyzed, such as whether a vehicle slowed (from lead car to truck) more than 5 mph and whether a vehicle was on or to the right of the edgeline when beside the truck.

## Roadway Variables Collected

Different geometric and traffic variables were collected for rural, two-lane conditions, including lane and shoulder width, degree of curve (or tangent), shoulder type (paved, gravel, or turf), speed limit, and intersection presence (with type of control). Data for most of the traffic and roadway variables were collected from more than one source for checking purposes, such as from field observations, photologs of the site, aerial photographs, and state department of transportation records. For each recorded opposing vehicle event, information was also recorded on the size of control truck (configuration, trailer length and width, and axle placement) and
on the type of opposing vehicle (small car, medium car, large car, pickup, van, station wagon, bus, small truck, or semitrailer).

Data for this study were desired only for rural roadways with speed limits of 45 and 55 mph under free-flow conditions (i.e., not within the influence of traffic signals). Thus, samples were excluded near signalized intersections, in towns and builtup areas, or where the speed limit was 40 mph or less. Several variables were collected but not used in the analysis, including shoulder type, because all shoulders in the sample sections were paved. The speed limit variable was not used in the analysis because speed was found to be strongly intercorrelated with other roadway variables used in the analysis.

Categories of several variables were grouped as a result of the sample sizes. Three categories remained with respect to curves and tangents:

- Lead car and truck on tangent,
- Lead car and truck on inside curve, and
- Lead car and truck on outside curve.

Samples were excluded for all other curve combinations (i.e., car on curve and truck on tangent for the same oncoming vehicle) because $t$-tests showed that the change in MOE for these conditions may be caused by geometric difference between curves and tangents and not by the presence of the truck. Also, sample sizes were small for these mixed combinations.

## Sample Sizes

After eliminating the data collected near intersections, on segments with speed limits of 40 mph or less, and on certain curve combinations, the remaining sample consisted of 3,330 observations with dry pavement conditions. A summary is presented in Table 1 of the sample sizes by truck type for certain geometric conditions. Overall, the largest sample was collected for the semi-40 (868 observations), followed by the semi-45 ( 756 observations), and semi-48 with axles back (703 observations), double-28 ( 648 observations), and semi-48 with axles forward ( 355 observations).

## Analysis Issues

Generally, the key issues addressed during analysis of the rural data included the following:

1. What roadway geometrics affect vehicle operations (i.e., vehicle speed and lateral placement) relative to large trucks on rural two-lane roads?
2. Were there differences in driving behavior between drivers of various truck sizes that could partially account for differences in the operation of oncoming traffic?
3. Do differences in MOEs exist between the semi-40, semi45 , semi- 48 with axles back, semi- 48 with axles up, and double-28, and if so, for what geometric conditions?
4. What were the most extreme reactions to the different control trucks by oncoming traffic and were any trends revealed?

A secondary purpose of the first issue was to help group data for analysis for the second and third issues. The fourth issue was addressed even though relatively few such events were expected to occur. Nonetheless, the number of extreme vehicle reactions (i.e., near-miss accidents) may be indicative of serious safety concerns for various truck types. For the third issue, more specific subquestions are addressed involving comparisons between pairs of truck types and sizes (e.g., comparing operations of a semi-40 versus semi-48).

## Issue 1: Roadway Geometrics That Affect Operations

 Relative to Large Trucks on Two-Lane, Rural RoadsLane width, shoulder width, presence of curvature, and to a lesser extent degree of curvature, all affect opposing vehicle operation relative to large trucks on two-lane, rural roads for the range of conditions tested.

One-way analysis of variance (ANOVA) tests were performed on roadway geometric variables using continuous MOEs for oncoming vehicles (a) speed change, (b) lateral placement at the trucks, and (c) lateral placement change. Results indicate that each variable tested has statistically significant changes for at least one MOE. Mean MOE values for some variable level show that there were generally only marginal changes in the means that were probably not operationally important. Thus, discrete MOEs aimed at identifying large operational changes were examined for lane width, shoulder width, and curve presence for the semi-48 with axles back. Results indicate that for the conditions tested, lane width was not an important factor with speed change MOEs, but that greater lane widths (i.e., 12- and 13-ft lanes) allow some opposing vehicles to move farther right when confronting a truck.

Shoulder width was a significant variable for both speed and lateral placement MOEs for the conditions tested. Speeds

TABLE 1 SUMMARY OF RURAL SAMPLE SIZES FOR VARIOUS TRUCK TYPES AND GEOMETRIC CONDITIONS

| Truck Type | Lane Width $=10$ and 11 ft |  |  |  | Lane Width $=12$ and 13 ft |  |  |  | Total |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: |
|  | $\begin{aligned} & \text { Shoulder Width }= \\ & 0 \text { to } 4 \mathrm{ft} \end{aligned}$ |  | Shoulder Width $\geq 4 \mathrm{ft}$ |  | $\begin{aligned} & \text { Shoulder Width }= \\ & 0 \text { to } 4 \mathrm{ft} \end{aligned}$ |  | Shoulder Width $\geq 4 \mathrm{ft}$ |  |  |
|  | Tangent | Curve | Tangent | Curve | Tangent | Curve | Tangent | Curve |  |
| Semi-40 | 135 | 50 | 141 | 64 | 128 | 64 | 141 | 145 | 868 |
| Semi-45 | 134 | 57 | 169 | 52 | 121 | 52 | 129 | 42 | 756 |
| Semi-48 (back) | 112 | 42 | 173 | 46 | 121 | 45 | 103 | 61 | 703 |
| Semi-48 (forward) | 108 | 48 | 12 | 6 | 78 | 32 | 37 | 34 | 355 |
| Double-28 | 116 | 43 | 95 | 36 | 111 | 58 | 68 | 121 | 648 |
| Total | $\overline{605}$ | $\overline{240}$ | $\overline{590}$ | $\overline{204}$ | $\overline{559}$ | $\overline{251}$ | $\overline{478}$ | $\overline{403}$ | $\overline{3,330}$ |

of oncoming vehicles increased by a greater amount between the lead car and the truck for wider shoulders than for narrow or absent shoulder conditions. Speed change MOE results showed that more vehicles increased speed between the lead car and the truck than decreased speed. This finding was likely caused by the tendency of oncoming motorists to decrease speed before passing the truck and then accelerate back to a comfortable free-flow speed as they passed the truck. Shoulders less than 4 ft wide tended to result in fewer vehicles on or over the edgeline and in combination with 12 - and $13-\mathrm{ft}$ lane widths resulted in more vehicles making lateral placement changes of 1 ft or more.
Presence of curvature affected the change in lateral placement, whereas direction of curvature (inside or outside curve) was not important for the conditions tested. The strong effect of curve presence on the three MOEs was also found. In all but one case of lane and shoulder width combinations, presence of curvature caused a change in the operation of oncoming traffic.

Degree of curvature did not affect the change in lateral placement (controlling for curve presence) and resulted in generally large speed reductions only at degrees of curvature greater than $7^{\circ}$. Direction of curve (i.e., inside or outside as faced by the opposing vehicle) did not have a significant effect on speed and lateral placement changes on the basis of $t$-tests.

In light of these findings for the effects of geometrics, results of one-way ANOVA tests on other variables (such as opposing vehicle size), and reexamination of available sample sizes, the approach used for the remaining analyses would include controls for lane width (10- or $11-\mathrm{ft}$ widths versus 12 - or 13 ft widths), shoulder width (shoulders less than 4 ft wide versus shoulders greater than 4 ft wide), and presence of curvature (yes versus no). The state variable (New Jersey or California sites) was controlled whenever appropriate. Finally, opposing vehicle type was controlled in the analyses involving lateral placement, on the basis of the one-way ANOVA tests.

## Issue 2: Differences in Driving Behavior That Could Partially Account for Differences in Operation

Significant differences were found in the lateral placement of some of the control trucks in the lane that could have had an effect on the operation of oncoming vehicles. Qualitative observation of the various control trucks in the field revealed that control-truck drivers made efforts to operate each truck type safely. For example, in operating the double-28 on narrow winding roads the driver was able to keep the vehicle within the lane with rare encroachments over the edgeline or centerline. However, the semi-48 with axles back had to be driven more cautiously on the narrow roads because of greater offtracking of the trailer when driving around curves. Thus, the driver often slowed the longer semitrailer considerably before approaching some of the curves. In some cases of curves to the left, on narrow lanes, for example, the driver of the semi-48 (with axles back) would encroach onto the right shoulder with the tractor to prevent the rear trailer from encroaching the centerline. The semi-40 and semi- 45 had little or no problem in normal driving on most of the routes.

Because qualitative observations suggested that the driver had to exercise added care with certain truck types, data were
analyzed to see the extent of differences between truck operations. Comparisons of truck types were made in terms of mean distances of the rear of the control truck from the centerline for instances when oncoming vehicles were directly beside the truck. This analysis was conducted only for more geometrically restrictive roadways (i.e., curve sections with lane widths of 10 or 11 ft and shoulders of 0 to 4 ft ) because those were considered to be the most critical situations. Average distances of the control trucks to the centerline for these situations were

- Semi-40-2.11 ft,
- Semi-45-2.04 ft,
- Semi-48 with axles back-2.06 ft,
- Semi-48 with axles forward- 1.84 ft , and
- Double-28-1.71 ft.

Even though results from offtracking models (1) show that the semi-48 offtracks more with axles back than with axles forward, the driver more than compensated for this because the semi-48 with axles back was generally farther from the centerline than the semi-48 with axles forward. In fact, the semi-40, semi-45, and semi-48 with axles back were each positioned about the same average distance ( 2.04 to 2.11 ft from the centerline) as they passed oncoming vehicles. The semi48 with axles forward and double- 28 were driven closer to the centerline, at 1.84 and 1.71 ft , respectively. Statistical $t$-tests were used to compare the means for each truck pair and to verify these differences, as presented in Table 2. Except for the comparison of the semi-48 with axles forward versus axles back (significance of 0.058 ), the double- 28 and semi- 48 with axles forward were positioned significantly closer to the centerline than were the other truck types ( 0.05 level).

An analysis was also conducted of the average clearance between each type of control truck and oncoming vehicles, as presented in Table 3. Average clearances for the truck types on the restrictive geometry were as follows:

- Semi-40-5.48 ft,
- Semi-45-5.38 ft,
- Semi-48 with axles back-5.54 ft,
- Semi-48 with axles forward- 5.00 ft , and
- Double-28-4.82 ft.

The $t$-tests were used to compare the means for pairs of trucks, which showed that the semi-48 with axles forward and double28 had significantly different clearances (i.e., less clearance distances) than the other three truck types.

In summary, drivers of the semi-48 with axles back compensated for added offtracking by the way in which they drove the vehicle. As a result, oncoming traffic was exposed to similar lane placements by the semi-40, semi-45, and semi-48 with axles back, and by lane placements that were closer to the centerline when passing the double- 28 and semi-48 with axles forward.

Issue 3a: Differences in MOEs Between Semi-40, Semi-45, Semi-48 with Axles Back, Semi-48 with Axles Forward, and Double-28

Differences in MOEs were observed between some of the truck types for a few of the geometric conditions tested. How-

TABLE 2 COMPARISON OF MEAN TRUCK DISTANCE TO CENTERLINE FOR CONTROL TRUCKS FOR SITES WITH RESTRICTIVE GEOMETRY ( $t$-TESTS)

| Truck Type Comparison | Mean Distance of truck to centerline (feet) |  | Calculated t-value | Degrees of freedom | $\begin{aligned} & \text { Two-tail } \\ & \text { proba- } \\ & \text { bility } \end{aligned}$ | Significance |  |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: |
|  | First truck type | Second truck type |  |  |  |  |  |
|  |  |  |  |  |  | . 05 | . 01 |
| Semi 40 vs. Semi 45 | 2.11 | 2.04 | 0.62 | 105 | 0.536 | No | No |
| Semi 40 vs. Semi 48 (back) | 2.11 | 2.06 | 0.38 | 92 | 0.706 | No | No |
| Semi 40 vs. Semi 48 (forward) | 2.11 | 1.84 | 2.48 | 96 | 0.015 | Yes | No |
| Semi 40 vs. Double | 2.11 | 1.71 | 3.14 | 91 | 0.002 | Yes | Yes |
| Semi 45 vs. Semi 48 (back) | 2.04 | 2.06 | 0.15 | 99 | 0.883 | No | No |
| Semi 45 vs. Semi 48 (forward) | 2.04 | 1.84 | 2.27 | 103 | 0.025 | Yes | No |
| Semi 45 vs. Double | 2.04 | 1.71 | 3.11 | 98 | 0.002 | Yes | Yes |
| Semi 48 (back) vs. (forward) | 2.06 | 1.84 | 1.92 | 90 | 0.058 | No | No |
| Semi 48 (back) vs. Double | 2.06 | 1.71 | 2.61 | 85 | 0.011 | Yes | No |
| Semi 48 (forward) vs. Double | 1.84 | 1.71 | 1.23 | 89 | 0.221 | No | No |

TABLE 3 COMPARISON BETWEEN TRUCK TYPES OF MEAN CLEARANCE BETWEEN CONTROL TRUCKS AND ONCOMING TRAFFIC FOR SITES WITH RESTRICTIVE GEOMETRY ( $t$-TESTS)

| Truck Type Comparison | Mean clearance between control truck and oncoming traffic (feet) |  | Calculated t-value | Degrees of freedom | $\begin{gathered} \text { Two-tail } \\ \text { proba- } \\ \text { bility } \\ \hline \end{gathered}$ | Significance |  |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: |
|  | First <br> truck type | Second truck type |  |  |  |  |  |
|  |  |  |  |  |  | . 05 | . 01 |
| Semi 40 vs. Semi 45 | 5.48 | 5.38 | 0.43 | 105 | 0.667 | No | No |
| Semi 40 vs. Semi 48 (back) | 5.48 | 5.54 | 0.27 | 92 | 0.788 | No | No |
| Semi 40 vs. Semi 48 (forward) | 5.48 | 5.00 | 2.24 | 96 | 0.028 | Yes | No |
| Semi 40 vs. Double | 5.48 | 4.82 | 2.74 | 91 | 0.007 | Yes | Yes |
| Semi 45 vs. Semi 48 (back) | 5.38 | 5.54 | 0.72 | 99 | 0.474 | No | No |
| Semi 45 vs. Semi 48 (forward) | 5.38 | 5.00 | 1.86 | 103 | 0.065 | No | No |
| Semi 45 vs. Double | 5.38 | 4.82 | 2.45 | 98 | 0.016 | Yes | No |
| Semi 48 (back) vs. (forward) | 5.54 | 5.00 | 2.64 | 90 | 0.010 | Yes | Yes |
| Semi 48 (back) vs. Double | 5.54 | 4.82 | 3.08 | 85 | 0.003 | Yes | Yes |
| Semi 48 (forward) vs. Double | 5.00 | 4.82 | 0.86 | 89 | 0.392 | No | No |

ever, for a majority of the situations tested, no significant differences were found. A summary of results of the truck type comparisons is presented in Table 4 for lateral placement changes of 1 ft or more. The $Z$-test for proportions was used for three MOEs.

- Proportion of lateral placement change $\geq 1 \mathrm{ft}$ from the centerline (Table 4). Only oncoming cars were used in this
comparison because prior analysis showed the insensitivity of oncoming trucks and buses to the control truck in terms of lateral placement.
- Proportion of oncoming vehicles (all types) that experience a speed reduction of 5 mph or more at the truck, compared with their speed while approaching the lead car.
- Proportion of oncoming cars that pass the control truck while on or over the edgeline.

TABLE 4 SUMMARY OF TRUCK COMPARISONS ON RURAL ROADS USING Z-TESTS FOR LATERAL PLACEMENT CHANGE OF $\geq 1 \mathrm{ft}$

| Comparison | Lane Width $=10$ and 11 ft . |  |  |  | Lane Width $=12$ and 13 ft . |  |  |  |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: |
|  | Shoulder <br> Width $=0$ to 4 ft . |  | Shoulder Width > 4 ft . |  | Shoulder <br> Width $=0$ to 4 ft . |  | Shoulder <br> Width > 4 ft . |  |
|  | Tangent | Curve | Tangent | Curve | Tangent | Curve | Tangent | Curve |
| Semi 40 vs. Semi 45 | 4 | 4 | D | $\bigcirc$ | C | 0 | $\bigcirc$ | C |
| Semi 40 vs. Semi 48 (back) | ) | ) | - | O | V |  | . |  |
| Semi 40 vs. Semi 48 (forward) | $\bigcirc$ |  |  |  |  |  |  |  |
| Semi 40 vs. Double | 4 |  |  |  |  |  |  |  |
| Semi 45 vs. Semi 48 (back) |  |  |  | 1 | 1 |  |  |  |
| Semi 45 vs. Semi 48 (forward) |  |  |  |  |  |  |  |  |
| Semi 45 vs. Double | ) |  | $\checkmark$ | D |  |  | $\bigcirc$ | ) |
| Semi 48 (back) vs. (forward) |  |  |  |  | A |  |  |  |
| Semi 48 (back) vs. Double | 1 |  | 0 | () | 1 | , | ( | C) |
| Semi 48 (forward) vs. Double | 1 | - |  |  | - | - |  |  |

[^5]Results of the MOE comparisons were produced for eight different combinations of geometric conditions on the basis of lane width ( 10 or 11 ft and 12 or 13 ft ), shoulder width (less than 4 ft , and 4 ft or greatcr) and curvature (tangent and curve). On the basis of lateral placement changes of $\geq 1 \mathrm{ft}$, the semi-45 faired significantly worse than the semi-40 for two geometric groups having narrow lanes and shoulders. The double caused significantly more oncoming vehicles to change their lateral placement than the semi-48 on narrow tangents. The semi-48 with axles back actually fared better than the semi-45 in two instances, which could be the result of the more conservative driving when the drivers operated the semi48 with axles back (i.e., because the truck was driven slightly farther from the centerline than the semi-45).

Rcsults of the analysis of edgeline encroachments revealed no significant differences between most truck types for a great majority of roadway situations. Significant differences existed in three situations, where (a) semi- 45 showed significantly more edgeline encroachments than the semi-40 (for wide lanes and shoulders on curves), (b) the semi-48 had more encroachments than the semi-40 (for narrow lanes and wide shoulders on tangents), and (c) the double had more encroachments than the semi-40 on narrow lanes with wide shoulders on tangents. Results of the $Z$-tests showed a mix of results with no clear trends.

A summary of the comparison of trucks using the analysis of covariance on continuous MOEs is presented in Table 5.

Results of testing with the continuous MOEs are given separately for two conditions.

1. Restrictive geometric segments (curves with lane widths of 10 or 11 ft , and shoulders of less than 4 ft ); and
2. Nonrestrictive geometrics segments (tangents with lane widths of 12 or 13 ft and shoulders of 4 ft or more).

For the analysis of covariance testing, control variables were used where appropriate to adjust mean values of the MOEs for the influence of such factors as state (New Jersey or California) and type of oncoming vehicle (car or truck).

A review of the results revealed several trends. For the $Z$ tests conducted on data from the least restrictive conditions (i.e., 12- or 13 -ft lanes with 4 -ft or wider shoulders), only 1 case out of 44 (with adequate data) showed a difference between truck types. However, applying Z-tests to data from the most restrictive geometry (i.e., lane widths of 10 or 11 ft and shoulders of less than 4 ft ), more truck comparisons ( 10 out of 60 ) were found to have significant differences between truck sizes.

Another finding was that more of the operational differences between truck types occur on tangents than on curves. Although somewhat unexpected, this finding tends to support an earlier finding that the lateral placement change did not change with increasing degrees of curvature. In other words, oncoming vehicles on tangent sections may be more likely to vary their lateral placement than on curves when passing a

TABLE 5 SUMMARY OF TRUCK COMPARISON OF VARIOUS MOEs ON RURAL ROADS USING ANALYSIS OF COVARIANCE (0.05 LEVEL)

| Truck Type Comparison | More Restrictive Geometrics |  |  |  | Less Restrictive Geometrics |  |  |  |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: |
|  | Lateral <br> Placement at truck | $\begin{array}{\|c\|} \hline \Delta \\ \text { Lateral } \\ \text { Placement } \\ \hline \hline \end{array}$ | Vehicle Speed at Truck | $\underset{\text { Vehic le }}{\Delta}$ Speed | Lateral Placement at Truck | Lateral <br> Placement | Vehicle Speed at Truck | Vehicle Speed |
| Semi 40 vs. Semi 45 | ) | ) | O | ) | D | O | 0 |  |
| Semi 40 vs. Semi 48 (back) | O |  |  | $\bigcirc$ |  |  |  | 1 |
| Semi 40 vs. Semi 48 (forward) |  |  |  |  |  | 1 |  |  |
| Semi 40 vs. Double |  |  |  |  |  |  |  | ) |
| Semi 45 vs. Semi 48 (back) |  | $\square$ |  |  |  |  |  |  |
| Semi 45 vs. Semi 48 (forward) | O |  |  |  |  | 4 |  |  |
| Semi 45 vs. Double |  |  |  | 4 |  | 0 |  |  |
| Semi 48 (back) vs. (forward) | $\bigcirc$ |  |  | O | $\dagger$ | 4 |  |  |
| Semi 48 (back) vs. Double | - | 4 |  | 0 | - | 0 |  | ) |
| Semi 48 (forward) vs. Double | ) | - | - | 4 | 0 | $\dagger$ | O | 0 |

O = No significant difference in MOE.
A = Significant increase in MOE for second truck type (i.e., the second truck had more effect than the first truck on the oncoming vehicle).
$\nabla=$ Significant decrease in MOE for second truck type (i.e., the second truck had less effect than the first truck on the oncoming vehicle).
large oncoming truck. This behavior may be caused by the larger effect of the curve than by the trucks on vehicle lateral placement.

## Issue 3b: Differences in MOEs Between Semi-40 and Semi-45

The semi- 45 was different than the semi-40 under some geometric conditions using some MOEs, but overall differences between the two truck types are not strongly supported. The semi-45 was associated with a significant increase in the proportion of vehicles with lateral placement changes of 1 ft or more in narrower lanes with narrower shoulders and an increase in the proportion of vehicles encroaching the edgeline on curved sections with wide lanes and shoulders. However, the semi-45 also was associated with a significantly lower proportion of $5-\mathrm{mph}$ speed changes than the semi- 40 . No differences in speed, speed change, lateral placement, or lateral placement change were found between the two vehicle types.

## Issue 3c: Differences in MOEs Between Semi-40 and Semi-48

Some significant differences were found in which the semi-48 affected oncoming traffic more than the semi-40. However, in a few other situations, the semi- 40 affected oncoming traffic more than the semi-48. Analysis of covariance results show that the semi-48 with axles forward was associated with sig-
nificantly greater lateral placement changes by oncoming vehicles for the less-restrictive geometrics condition. Oncoming motorists moved laterally away from the semi-48 with axles forward an average of 0.52 ft compared with 0.07 ft for the semi-40. Significantly more vehicles were observed over the edgeline on 10 - and $11-\mathrm{ft}$ lane tangent sections with wide shoulders when passing the semi- 48 with axles back than the semi-40.
By contrast, several instances were found that showed that the semi-40 affected oncoming traffic more than the semi-48. Several instances were found of significantly lower proportions of vehicles experiencing speed changes of 5 mph for the semi-48 with axles back compared to the semi-40. Although somewhat contrary to expected results, these findings could be caused partly by different truck operation and lane placement of the truck types.

## Issue 3d: Differences in MOEs Between Semi-40 and Double-28

Of the geometric and MOE conditions tested, only two showed any significant differences between the semi-40 and the double-28. First, for tangent sections with narrower lanes and shoulders, a significantly higher proportion of oncoming vehicles moved laterally 1 ft or more when passing the double, compared with the semi-40. Second, a significantly higher proportion of vehicles encroached the edgeline when passing the double- 28 compared with the semi- 40 for $10-$ and $11-\mathrm{ft}$ tangent sections with wide shoulders. However, little or no
differences were found in oncoming vehicle operations between the double-28 and semi-40 for most of the two-lane roadway conditions tested. Surprisingly, few operational differences resulted, particularly because the double-28 was driven closer to the centerline than the semi-40.

## Issue 3e: Differences in MOEs Between Semi-45 and Semi-48

Significant differences were found in several cases. Like the comparison of the semi-40 and the semi-48 in Issue 3c, the comparison of the semi-45 and semi-48 resulted in one truck's affecting traffic significantly in some cases and the other truck's affecting traffic significantly in other cases. Four geometric conditions were found in which the semi-48 (axles back in most cases) caused a higher proportion of speed changes of 5 mph or greater compared with the semi- 45 . However, the semi-48 was associated with a lower proportion of oncoming vehicles with lateral placement changes of 1 ft or greater for two geometric conditions. Also, a reduction in the average change in lateral placement was found under more restrictive geometric conditions for the semi-48 with axles back, compared with the semi-45. One explanation for these unexpected lateral placement results is the differences in the manner the trucks were driven, as discussed in Issue 2.

## Issue 3f: Differences in MOEs Between Semi-45 and Double-28

Significant differences were found for a few geometric conditions. Analysis of covariance revealed significantly greater vehicle speed changes for the double-28 in more restrictive geometric conditions. This finding was consistent with other results that revealed a significantly higher proportion of oncoming vehicles with speed changes of 5 mph or greater for the double-28, compared with the semi-45 in cases of narrower roads on curves. An explanation may be partly found in the double- 28 's being driven closer to the centerline than the semi-45.

The proportion of vehicles with a change in lateral placement of 1 ft or greater was significantly less for doubles than for the semi-45 on tangents with narrow lanes and wide shoulders. However, no differences in average lateral placement change between the semi- 45 and double- 28 were found by using the analysis of covariance.

In summary, evidence exists that oncoming motorists may slow down more for doubles than for the semi-45, possibly because they see a longer truck and expect a problem. However, the fact that oncoming motorists do not change lateral placement when beside the double-28 may show that the drivers perceived no need for evasive action, possibly because the offtracking of the double- 28 on the two-lane roadways rarely presented much of a problem for oncoming traffic.

## Issue 3g: Differences in MOEs Between Semi-48 and Double-28

Although differences in MOEs were found in several cases, the results are mixed. Few lateral placement changes of 1 ft
or greater for oncoming traffic were found for the semi-48 in three cases (all narrow shoulder conditions). However, the average lateral placement change was significantly lower for the double- 28 in one case and lower for the semi-48 in another. Average vehicle speed changes were greater with the double in one case of more restrictive geometrics, whereas the double- 28 was associated with a lower proportion of speed changes of 5 mph or more for tangent roadways with wider lanes and narrower shoulders.

In summary, the inconsistent results in operations between the double and semi- 48 preclude identification of one type as clearly a greater operational problem. The manner of operation of these two truck types was also considered to be a possible factor in the mixed results, as discussed earlier.

## Issue 3h: Differences in MOEs Between Semi-48 Axles Forward and Semi-48 Axles Back

In three of four situations where significant differences were found, the semi- 48 with axles forward was shown to have greater operational effects on oncoming traffic than the semi48 with axles back. This finding can be explained by the manner in which the two control trucks were operated. As discussed, the semi-48 with axles back was generally driven a greater distance from the centerline than the semi-48 with axles forward ( 2.06 to 1.84 ft average distance, respectively, which is a significant difference at the 0.10 confidence level but not at the 0.05 level).

## Issue 4: Reactions to Different Control Trucks by Oncoming Traffic

In a few cases, oncoming traffic was affected considerably by the control trucks, particularly by the semi-48 and the double28. The previous analyses involved average vehicle operations for various sample sets. However, efforts were also made to review extreme reactions to the various control trucks by oncoming traffic, as a possible indication of near-miss accidents.

Four operational MOEs were analyzed including

- Change in lateral placement (ft) of oncoming vehicles (i.e., how far an oncoming vehicle moved over in the lane in response to the oncoming control truck);
- Change in speed (mph) of oncoming vehicles (for those vehicles that slowed down in response to the control truck);
- Clearance ( ft ) between control truck and oncoming vehicle; and
- Distance of the vehicle to the right of the edgeline.

For each of these measures, the extreme value (maximum or minimum) and the first and third percentile values were determined for each type of control truck and are presented in Table 6 for all sites. The largest changes in lateral placement were a $5.5-\mathrm{ft}$ movement by a motorist in response to the double-28, and a $4.8-\mathrm{ft}$ movement by a motorist for the semi48 with the axles back. However, first percentile values were nearly identical between truck types, ranging from 2.3 to 2.6 ft (highest for the semi-48 with axles back). At the third percentile, values for different trucks were also close and

TABLE 6 SUMMARY OF THE EXTREME REACTIONS TO CONTROL TRUCKS BY ONCOMING VEHICLES

| Operational Measure | Measure Value | Truck Type |  |  |  |  |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: |
|  |  | Semi 40 | Semi 45 | Semi 48 (axles back) | Semi 48 (axies forward) | Double |
| Change in lateral | Maximum | 3.2 | 3.3 | 4.8 | 3.5 | 5.5 |
| placement of on- | 1 Percentile | 2.3 | 2.3 | 2.6 | 2.4 | 2.3 |
| coming vehicles (feet) | 3 Percentile | 1.6 | 1.6 | 1.7 | 1.7 | 1.9 |
| Change in speed of | Maximum | 20.0 | 13.0 | 19.0 | 21.0 | 24.0 |
| oncoming vehicles | 1 Percentile | 9.0 | 7.0 | 12.0 | 12.8 | 8.0 |
| (mph) | 3 Percentile | 6.0 | 4.0 | 5.0 | 8.0 | 5.5 |
| Clearance between | Maximum | 2.9 | 2.9 | 2.8 | 2.9 | 2.5 |
| control truck and | 1 Percentile | 3.8 | 3.4 | 3.4 | 3.1 | 3.3 |
| ancoming vehicles (feet) | 3 Percentile | 4.3 | 4.1 | 3.8 | 3.5 | 3.8 |
| Distance of | Maximum | 3.9 | 4.6 | 1.7 | 2.1 | 2.6 |
| oncoming vehicles | 1 Percentile | 1.2 | 1.2 | 1.3 | 1.0 | 1.2 |
| beyond edgeline (feet) | 3 Percentile | 0.3 | 0.4 | 0.5 | 0.0 | 0.6 |

ranged from 1.6 ft (for the semi-40 and semi-45) to 1.9 ft (for the double-28).

Maximum change in speed came in response to the double$28(24 \mathrm{mph})$ and the semi- 48 with axles back ( 21 mph ). The semi- 40 had an unexpectedly high $20-\mathrm{mph}$ speed reduction in one case. At the first percentile, the greatest speed reduction came from the semi- 48 , with 12.8 - and $12.0-\mathrm{mph}$ speed changes in response to the axles forward and axles back condition, respectively.

Minimum clearances between control trucks and oncoming vehicles ranged from 2.5 ft (double-28) to 2.9 ft (three truck types). For the first and third percentile levels, clearances were generally slightly less for the semi-48 and double- 28 than for the semi-45 and semi-40.

Maximum edgeline encroachments were found for the semi$45(4.6 \mathrm{ft})$ and semi-40 (3.9 ft). However, the first percentile values were consistent with other MOEs with the greatest value ( 1.3 ft ) for the semi-48 with axles back. At the third percentile, the double- 28 and semi- 48 with axles back caused the highest values of 0.6 and 0.5 ft , respectively.

This analysis showed that there are isolated extreme operational incidents that occur because of oncoming vehicles passing large trucks. Keeping in mind that a single maximum or minimum value may be influenced by many factors other than truck type, the trend to greater extreme values (i.e., at the first and third percentile levels) is indicated for the semi-48 and double-28. However, differences at these levels are generally small and may be within the range of the standard error of the data.

## SUMMARY AND CONCLUSIONS

The ability of various truck configurations to negotiate roads and streets with restrictive geometry was studied in addition
to the effects of such trucks on traffic operations and safety of such roads and streets. Truck sizes included truck-tractor semitrailers with trailer lengths of 40,45 , and 48 ft and trailer widths of 96 and 102 in ., and twin-trailer combinations with 28 -ft trailers.

Also included was a review of literature and an analysis of offtracking of truck sizes of concern. Test sites consisted of rural two-lane roads in New Jersey and California with lane widths of 10 to 13 ft , shoulder widths ranging from 0 to approximately 10 ft , and different types of horizontal alignment (tangents, gentle curves, severe curves).

Control trucks were used for testing at the sites (i.e., staged experiments using a professional driver and rented truck tractors and trailers). Statistical testing using $t$-test, analysis of variance and covariance, $Z$-test for proportions, and other tests were conducted to compare operational differences between the various truck types.

## Findings

Roadway geometrics that affect vehicle operations relative to large trucks on rural, two-lane roads include lane width, shoulder width, and presence of curvature. Wider (12 or 13 ft ) lanes allow opposing vehicles to move farther right in encounters with trucks and fewer vehicles cross the edgeline with wider lanes. Wider ( 4 ft or greater) shoulders generally allowed opposing vehicles to accelerate to regain their freeflow speed, move farther to the right, and cross the edgeline more frequently while passing the truck. Presence of curvature usually meant greater operational changes (i.e., speed changes) and undesirable maneuvers by opposing vehicles. Degree of curvature had little effect on lateral-placement MOEs over the ranges tested, but large degrees of curvature (i.e., $7^{\circ}$ to $15^{\circ}$ ) did cause opposing vehicles to slow while passing large trucks.

Drivers of the control trucks compensated for the greater offtracking of the semi- 48 with axles back by driving farther from the centerline than the semi-48 with axles forward or the double-28. In fact, no differences were found in average distance to the centerline or in clearance between the trucks and opposing vehicles between the semi- 48 with axles back, semi-40, and semi-45. Driver skill and caution on rural roads seemed important in the operation of vehicles that interact with the large trucks.

Some statistically significant differences in MOEs were found between the larger trucks (semi-48 and double-28) and smaller trucks (semi-40 and semi-45). However, these differences were numerically quite small. Oncoming motorists moved away from the semi-48 with axles forward or the double-28 more than the semi-40 and strayed over the edgeline for the semi48 with axles back or the double- 28 more often than for the semi-40. The semi-48 and double-28 also caused motorists to make 5 -mph (or more) changes in speed more often than the semi-45. However, in general, the results showed many situations in which no significant operational differences existed between truck types. Also, significant differences were found in a few cases with the smaller truck in the comparison causing greater operational changes than the larger truck. For the range of conditions tested, other factors such as driver skill (as evidenced by handling of the truck) and roadway geometrics seemed to affect the operations of oncoming vehicles on two-lane, rural roads as much or more than truck type.

Analysis of extreme values for certain MOEs (as a measure of near-miss accidents) showed that a few drastic speed changes and lateral placement changes did occur by oncoming vehicles when passing large trucks. The semi- 48 with axles either forward or back and the double-28 were generally associated with more extreme changes by oncoming motorists than the semi-40 and semi-45.

## Implications of Study Results

The results have several implications with respect to operational effects of larger trucks. The literature review and offtracking data indicate that truck width is a less important issue than truck length for the trucks of interest. All field testing was thus conducted with emphasis on truck length and configuration. Placement of the rear axles was also studied in the case of the semi-48.

Truck drivers often handle the larger trucks (i.e., semi-48s and double-28s) differently than the smaller trucks (i.e., semi40 s and semi-45s). Different handling can at least partly compensate for increased offtracking and length of the larger trucks, and may mean fewer operational problems than might otherwise be expected.

Test sites used were selected to be only somewhat restrictive because severe encroachments were not desirable in field testing. However, some of the sites approached the limits of geometric conditions at which more effects of the larger trucks became evident. In particular, the semi-48 with axles back caused more operational problems on rural two-lane roads with narrow lanes and narrow shoulders.

A variety of test conditions and MOEs were used. In spite of this variety, the results do not provide sufficient information for recommending blanket regulations for larger trucks.

However, it was evident that combinations of geometric conditions at a site must be considered before establishing truck restrictions. For example, $12-\mathrm{ft}$ lanes combined with sharp horizontal curves on a rural, two-lane road can lead to more operational problems for larger trucks than on 11 -ft lanes on a tangent section.

## Recommendations for Future Research

Tests were primarily conducted under ideal conditions. Most of the results were based on two highly experienced drivers, knowledgeable of the experiment purpose, operating trucks in good condition over known routes with dry pavement during the day. Thus, a need remains for knowledge of large truck operation in the general traffic stream under less-thanideal conditions. Operational problems associated with larger trucks may be caused by inexperienced or impaired drivers with faulty equipment in severe weather, and these and other less-than-ideal conditions should be examined.

Several other issues exist involving larger trucks that could use additional scrutiny. Ranges of geometric conditions not covered in this study remain an issue because they pertain to inclusion in the National Truck Network. For example, samedirection passing of wider and longer trucks on narrow, multilane highways has emerged as a concern. Also, longer semitrailers (i.e., $53-\mathrm{ft}$ ) are now allowed by most states with unknown effects on operations and safety.

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# Assessment of Current Speed Zoning Criteria 

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As early as 1947, studies concluded that the majority of drivers ignore speed limits and drive at speeds that they believe are safe and reasonable. Since then, some studies have supported this conclusion whereas others indicated that speed limits do affect travel speeds in varying degrees. In an FHWA-sponsored assessment of current speed zoning criteria, speed and accident data were collected at 50 locations, both urban and rural, in four states on roadways with posted speed limits ranging from 25 to 55 mph . These data were analyzed to determine travel speed characteristics, compliance with posted speed limits, and the point of minimum accident risk. Significant findings were as follows: Mean speeds exceeded posted speed limits by 1 to 8 mph ; 85thpercentile speeds ranged from 6 to 14 mph over the posted speed limit, or 4 to 7 mph over the mean speed; the majority ( 70.2 percent) of free-flow drivers observed did not comply with posted speed limits; in general, 85 percent compliance was achieved at speeds 10 mph over the posted speed limit; accident rates for the $25-\mathrm{mph}$ zones were consistently much higher than for any of the other zones; and the speed at which accident risk was minimized occurred at the 90th percentile of the travel speeds observed.

The practice of establishing speed limits, or speed zoning, began early in the history of motorized travel when officials realized that excessive speed could result in damage and injury to others. The first speed limit in the United States was enacted in Connecticut in 1901 and since that time the evolution of speed limits in this country has become both complex and controversial. As early as 1947 , studies have concluded that the majority of drivers ignore speed limits and drive at speeds that are believed safe and reasonable (1). Since then, some studies have supported this conclusion whereas other research indicated speed limits do affect travel speeds in varying degrees (2).

Perhaps one reason for the lack of consensus on the effect of speed limits is the lack of uniformity in establishing speed limits. Speed zoning is generally defined as the establishment of safe and reasonable speed limits. Although safe speed is difficult to define, reasonable speed is nearly impossible for all drivers to agree on. Broad interpretations of these terms combined with the lack of sound engineering knowledge has led to use of a wide variety of regulations and procedures posting speed limits.

Lack of uniformity in speed zoning among jurisdictions creates problems for motorists, law enforcement officials, and judges. If posted speed limits are unreasonably low, the majority of drivers become technical violators of the law, which places law enforcement officials and judges in the position of arbi-
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trarily selecting violators. This process also produces criticism of highway officials responsible for posting speed limits and can often lead to concerned citizens and legislators taking an active role in speed zoning decisions (3).

In an FHWA-sponsored assessment of current speed zoning criteria, speed and accident data were collected at 50 locations, both urban and rural, in four states on roadways with posted speed limits ranging from 25 to 55 mph (4). These data were analyzed to determine travel speed characteristics, compliance with posted speed limits, and the point of minimum accident risk.

## SITE SELECTION

Although travel speed characteristics may vary from one state to another, an attempt was made to select states believed to be representative of travel speeds in a particular region of the country. This result was accomplished by dividing the nation into four geographic regions (southwest, northeast, midwest, and west) and selecting two states from each region (one primary and one alternate) on the basis of the following criteria:

- The national maximum speed limit (NMSL) data base had to contain 3 years of vehicle speed data for the control sampling locations;
- The accident data base had to be computerized and accessible by highway location; and
- The accident data reported by each state had to contain, at a minimum, estimated travel speed, posted speed limit, violations cited, time of day, day of week, severity, type of collision, and number of vehicles involved.

On the basis of these criteria, the four states selected were North Carolina, Delaware, Colorado, and Arizona.

Selection of roadway segments within each state for which speed and accident data were collected began with the stratification of sites by area type, roadway type, and speed limit. On the basis of national daily vehicle miles of travel (DVMT) obtained from the Higliway Performance Monitoring System (HPMS) data, sites were selected in 11 cells (8 urban and 3 rural cells) as presented in Table 1. The 11 cells chosen were multilane and two-lane cells with the largest DVMT value. Included were five multilane sites and six two-lane sites in each state.

Sites were selected in each of the four states using a data tape containing HPMS roadway segments and their characteristics. Segments were first stratified into the 11 cells identified for data collection in Table 1. A list of segments within

TABLE 1 STRATIFICATION OF STUDY SITES BY AREA TYPE, ROADWAY TYPE, AND SPEED LIMIT (NATIONAL DVMT)

| Speed <br> Roadway Type | Area Type |  |  |  |  |  |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: |
|  | Rural |  | Small Urban |  | Large Urban |  |
|  | 25/30/35 | 40/45/50 | 25/30/35 | 40/45/50 | 25/30/35 | 40/45/50 |
| Freeway | 374 | 1,320 | 403 | 2,323 | 5,663 | 62,781 |
| Multilane | 9,182 | 29,716 | 34,354 | 26,783 | 276,018 | 259,248 |
| Two Lane | 67,770 | 179,712 | 76,700 | 27,575 | 330,968 | 151,328 |

each cell was then compiled for each state showing the HPMS number, length of the segment, location (county), and average daily traffic.
In order to make data collection as efficient as possible, sites were selected in one or two concentrated areas within each state. A large urban city was selected as the center of each area, and the surrounding counties falling within a 100 to 200 -mi radius were included to complete the cluster of counties in each state from which the sites were selected. The section length for some of the segments identified by HPMS was small (less than 0.3 mi in many cases). Data collection required the vehicles to be traveling at free-flow speeds. In order to achieve this goal, no segments with lengths of less than 0.4 mi were selected.
Using the list of available HPMS sections and a random numbers table, one primary site and two alternate sites were selected for each cell in each state. Alternate sites were used in cases in which the primary site had been physically changed or could not be used at the time of data collection.
In addition to the non- $55-\mathrm{mph}$ sites selected for data collection, six $55-\mathrm{mph}$ sites were selected from the control sampling locations used in monitoring NMSL. Speed data are collected four times a year at each of these locations by the state transportation personnel. Of the six rural sites selected, four were on two-lane roads and two were on multilane highways.

## DATA COLLECTION

## Speed Measurement

Speed data were collected at a total of 50 sites (44 non-55mph sites and six $55-\mathrm{mph}$ sites). At each location, 24 hr of vehicle speed data was collected using an International Road Dynamics 1040 Traffic Statistics Recorder (TSR) and inductive loops. These loops were either in rubber mats fabricated by the project team or were permanent loops used by the state at the NMSL locations. When the loop mats were used, the equipment was deployed at a location within each segment determined by three factors: section representation, adequate sight distance, and availability of permanent structures to which the equipment could be secured.

The first factor, representation, was the most important. A location had to be selected where vehicles traveled at freeflow speeds that were representative of speeds throughout the
section. This criterion meant avoiding locations close to intersections or locations governed by advisory speed zones. Once a deployment location was found, the mats were placed in the center of each lane at a distance of 16 ft from leading edge to leading edge.

With the mats and lead wires securely in place, the accuracy of the unit was checked by comparing speeds obtained with a hand-held radar unit to speeds obtained by the TSR. The TSR recorded raw vehicle data in terms of speed, length, and time with respect to the lane of travel for each vehicle. Actual number of vehicles (raw counts), usable number of vehicles (actual counts), number of passing vehicles, and number of loop errors for each hour the unit was operating were recorded in the TSR statistics table.
Usable vehicles were defined as vehicles with a length and speed. If a vehicle did not cross both mats in a lane, an upstream only or downstream only reading would be recorded depending on which mat was missed. These vehicles were stored under the raw vehicle count but not under the usable vehicle count. When a loop error occurred, the upstream only or downstream only reading was also recorded and this information was used to determine the percentage of usable vehicles with respect to the raw vehicle counts and to determine whether the data for the 24 -hr period were acceptable. If this percentage was less than 70 percent, another $24-\mathrm{hr}$ period of data was collected before leaving the site.
At the conclusion of the 24 -hr period, the equipment was removed from the roadway. The TSR statistics table was checked to determine if the 70 percent threshold had been achieved. If this percentage had not been obtained, measures were taken to determine the faulty loops or broken lead wires, and the equipment was removed, repaired, and replaced in the roadway for another $24-\mathrm{hr}$ period. If the threshold had been obtained, the equipment was removed from the roadway. Raw vehicle data and TSR statistics table were then loaded into a laptop computer for analysis.

Raw vehicle data from each site were analyzed using a program developed by FHWA. From this program, descriptive speed statistics, percentiles, pace, compliance data, and other factors were obtained. Results from this program for each of the sites were combined and used in the overall analysis.

## Site Characteristics

In addition to speed data collected at each of the selected roadway segments, several other geometric and traffic
characteristics collected were

- Development type (commercial, residential, etc.),
- Number of intersections (signalized and unsignalized),
- Number of intersecting roadways,
- Horizontal curvature (none, moderate, severe),
- Terrain (flat, rolling, mountainous),
- Median type and width,
- Posted speed limit,
- Ádvisory speeds and speed zones,
- Lane width, and
- Shoulder type and width.

Each item was collected for the entire length of each segment. For purposes of collecting these data and the accident data, each segment was extended in both directions beyond the original mileposts defining the HPMS section as long as the characteristics of the site remained constant.
Annual average daily traffic (AADT) volume for each roadway segment was obtained from the state. This information was used in combination with the accident data to develop accident rates.

## Accident Data

Three years of accident data were collected from the states for each of the roadway segments where speed data were collected. Variables obtained were

- Route,
- Milepost,
- Date,
- Time of day,
- Day of week,
- Number of injuries,
- Number of fatalities,
- Speed involved (yes or no),
- Collision type,
- Vehicle type,
- Estimated travel speed,
- Weather conditions,
- Surface conditions,
- Driver condition,
- Accident severity, and
- Intersection accident (yes or no).

Of the variables listed, estimated travel speed was the only one not obtained in all states. Although the value was recorded on the accident report in the field by the investigating officer, data were not available from the computerized accident file in Arizona or Delaware.

## RESULTS

## Travel Speed Characteristics

## General

Rural, small urban, and urban stratification used in identifying sites from the HPMS data base was eliminated on the basis
of observations of the field data collection team. Some urban sites appeared to be more rural than urban and vice-versa on the basis of factors such as vehicle volumes, development type, intersection and driveway density, etc. Thus, giving results of the analysis in terms of rural versus urban would have been not only difficult, but misleading. Therefore, results are given in terms of posted speed limit and road type (number of lanes). Geometric and site attributes collected in the field by the project team were then used to better define area type influences on travel speed. Distribution of the 50 sites (by number of travel lanes and posted speed limit) used in the analysis are presented in Table 2.
Figure 1 shows the box plots of travel speed percentiles for each speed zone. Travel speed percentiles used for each plot are $5,15,50,85$, and 95 . Plots express the normality of each speed zone data set because the 5 th, 15 th, 85 th, and 95 th percentiles are symmetrical about the 50 th percentile. As shown by these plots, variations across speed zones are similar, indicating a homogeneous sample.

TABLE 2 DISTRIBUTION OF STUDY SITES BY NUMBER OF TRAVEL LANES AND POSTED SPEED LIMIT



POSTED SPEED LIMT

FIGURE 1 Box plots of speed percentiles for each speed zone.

Examining speed variance shows the standard deviation to range from 5.44 mph for $25-\mathrm{mph}$ zones to 7.59 mph for $50-$ mph zones. In order to compare the variance between groups of sites within different speed zones, the coefficient of variation was calculated for each speed limit group and is presented in Table 3. This measure of relative variation indicates no consistent pattern with the exception of the $45-$, 50 -, and $55-\mathrm{mph}$ zones having the smallest values indicating a lower degree of spread in the distribution.

## Two-Lane Versus Multilane Roadways

Comparison between free-flow mean speeds for two-lane roads versus multilane roads showed no observable differences except for the $30-\mathrm{mph}$ zone. However, in this case, the multilane cell contained only one site. As shown in Figure 2, the means exceeded the posted speed limit across all speed zones by 1

TABLE 3 COEFFICIENT OF VARIATION FOR EACH SPEED LIMIT GROUP

| Speed <br> Limit | Standard <br> Deviation | Coefficient <br> of Variation |
| :---: | :---: | :---: |
| 25 | 5.44 | $17.6 \%$ |
| 30 | 5.73 | $15.7 \%$ |
| 35 | 6.58 | $17.1 \%$ |
| 40 | 7.30 | $17.5 \%$ |
| 45 | 6.88 | $14.2 \%$ |
| 50 | 7.59 | $14.7 \%$ |
| 55 | 6.70 | $11.8 \%$ |



FIGURE 2 Comparison of mean speeds with posted speed limit by speed zone.
to 4 mph with the exception of the 25 - and $30-\mathrm{mph}$ speed zones in which a difference of 8 mph was observed. The 85 thpercentile speeds, shown in Figure 3, ranged from 6 to 14 mph over the posted speed limit or 4 to 7 mph over the mean speed. Extremes for the mean and 85 th-percentile speeds are presented in Table 4. In no case was the 85th-percentile speed less than the posted speed limit.

## Cars Versus Trucks

Comparison between car and truck speed characteristics showed that cars travel 1 to 5 mph faster than trucks for all speed


FIGURE 3 Eighty-fifth-percentile speeds.

TABLE 4 EXTREMES FOR MEAN AND 85thPERCENTILE SPEEDS

| Speeds > mean |  |  |  |  |
| :---: | :---: | :---: | :---: | :--- |
| Site <br> No. | Speed <br> Limit | No. of <br> Lanes | Mean <br> (mph) | 85 th <br> $($ mph $)$ |
| $8270 B$ | 30 | 2 | 44.2 | 51.0 |
| 242 | 30 | 2 | 41.3 | 47.8 |
| 11829 | 35 | 4 | 44.8 | 51.9 |
| Speeds < mean |  |  |  |  |
| Site | Speed | No. of | Mean | 85 th \% |
| No. | Limit | Lanes | (mph) | $($ mph $)$ |
| 304 | 35 | 4 | 31.0 | 36.1 |
| 361 | 50 | 2 | 46.7 | 54.0 |
| 41 | 30 | 2 | 29.9 | 34.3 |

zones (see Table 5). For both cars and trucks, 85th-percentile speeds were 4 to 7 mph greater than the mean speed. The largest difference between car and truck speeds for an individual site was 9.8 mph , whereas the smallest observed difference was zero.

## Day Versus Night

Free-flow mean and 85th-percentile speeds presented in Table 6 for daytime, nighttime, and dawn and dusk indicated a 0 - to 3 -mph difference. However, no consistent increase or decrease in speeds was observed on the basis of time of day across all speed classes. The largest observed difference for an individual site was a night speed of 12.5 mph below the dawn and dusk speed. Several sites had speed differences less than 0.5 mph .

## Compliance with Posted Speed Limits

## General

Throughout the analysis, compliance was treated in terms of the percentage of free-flow vehicles exceeding the posted speed limit, i.e., those vehicles not in compliance with the law. In general, data from the 50 sites revealed that the majority of drivers ( 70.2 percent) did not comply with posted speed limits. Noncompliance by site ranged from a low of 32 percent to a high of 97 percent with the exception of one site where noncompliance was only 1 percent. This finding remained generally consistent throughout the analysis, regardless of the factors or combination of factors examined.

Figure 4 shows percent noncompliance by posted speed limit. The number at the top of each bar is the percent exceed-


FIGURE 4 Percentage of noncompliance by posted speed limit.
ing the limit; the next number is the percent exceeding the speed limit by more than 5 mph ; the next by more than 10 mph ; and the bottom number by more than 15 mph . Overall, the percent of drivers exceeding posted speed limits by more than 5 mph was 40.8 percent; by more than $10 \mathrm{mph}, 16.8$ percent; and by more than $15 \mathrm{mph}, 5.4$ percent. If the $85 \mathrm{th}-$

TABLE 5 CAR AND TRUCK SPEED CHARACTERISTICS

| Speed <br> Limit <br> (mph) | Mean speeds (mph) |  | 85th \%tile Speeds (mph) |  |
| :---: | :---: | :---: | :---: | :---: |
|  | Car | Truck | Car | Truck |
| 25 | 31.1 | 29.0 | 36.2 | 33.6 |
| 30 | 36.6 | 32.6 | 42.2 | 39.1 |
| 35 | 38.6 | 36.6 | 44.6 | 41.3 |
| 40 | 41.8 | 38.4 | 48.4 | 44.9 |
| 45 | 48.6 | 44.4 | 54.6 | 51.1 |
| 50 | 51.6 | 48.1 | 58.6 | 54.5 |
| 55 | 56.3 | 53.9 | 62.3 | 60.5 |

TABLE 6 FREE-FLOW MEAN AND 85th-PERCENTILE SPEEDS FOR DAYTIME, NIGHTTIME, AND DAWN OR DUSK CONDITIONS

| Speed <br> Limit <br> (mph) | Mean Speeds (mph) |  |  | 85 th \%tile Speeds (mph) |  |  |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: |
|  | Day | Night | Dawn/Dusk | Day | Night | Dawn/Dusk |
| 25 | 30.8 | 30.8 | 31.1 | 36.1 | 35.7 | 35.9 |
| 30 | 36.6 | 34.4 | 36.7 | 42.1 | 39.2 | 41.9 |
| 35 | 38.5 | 38.9 | 38.6 | 44.4 | 44.9 | 44.4 |
| 40 | 41.4 | 39.8 | 41.2 | 48.1 | 45.9 | 48.2 |
| 45 | 48.5 | 49.3 | 48.7 | 54.4 | 55.0 | 54.8 |
| 50 | 51.3 | 51.6 | 51.8 | 58.1 | 56.9 | 58.2 |
| 55 | 56.1 | 56.8 | 56.2 | 62.2 | 61.9 | 61.8 |

percentile speed criteria for establishing speed zones was carried over to compliance, the data indicated that, in general, 85 percent compliance was achieved at speeds 10 mph over the posted speed limit.

Noncompliance ranged from a low of 6.2 percent in 40 mph zones to a high of 83.4 percent in $25-\mathrm{mph}$ zones. The only discernible pattern was with the percentages exceeding the limits by more than 10 and 15 mph . Percentage of noncompliance at each of these levels at speed limits of 40 mph and greater was about half that of the noncompliance for sites with speed limits under 40 mph , which may indicate some reluctance by drivers to speed excessively on higher speed roadways.

## Two-Lane Versus Multilane Roadways

Data were next examined for two-lane versus multilane roadways for each posted speed limit. Table 7 presents the percentages of drivers exceeding the posted speed limit, and then exceeding the limit by more than 5,10 , and 15 mph . The comparison of noncompliance for two-lane roads versus multilane roads for $25-$ and $30-\mathrm{mph}$ speed zones is not meaningful because there was only one multilane site in each case. In $35-$, $45-$ - 50 -, and $55-\mathrm{mph}$ zones, noncompliance on multilane roads was higher, whereas in $40-\mathrm{mph}$ zones, multilane noncompliance was lower in all categories.

## Cars Versus Trucks

As presented in Table 8, noncompliance was higher for cars than for trucks at all levels. Car noncompliance ranged from a low of 63.2 percent in $40-\mathrm{mph}$ zones to a high of 83.7 percent in $25-\mathrm{mph}$ zones. Noncompliance for trucks, which made up an average of 1.38 percent of the free-flow traffic stream,
ranged from a low of 40.6 percent in $40-\mathrm{mph}$ zones to a high of 70.0 percent in $25-\mathrm{mph}$ zones. When classified by road type, i.e., two-lane versus multilane, the results were essentially the same. Trucks had a lower noncompliance percentage at all levels in all speed zones. In general, cars exhibited a higher noncompliance percentage on two-lane roads than on multilane roads. With respect to trucks, the results were mixed for two-lane versus multilane roads in all speed zones.

## Day Versus Night

Data were classified by time of day into three categoriesday, night, and dawn or dusk. As shown in Figure 5, some differences in noncompliance percentages were evident. Significant differences occurred at sites posted as 30 and 40 mph . Percentage of drivers not complying with the speed limit at these locations during the night hours decreased by 9 to 11 percent over the daytime noncompliance rate. However, by examining the percentage of drivers excessively over the posted speed limit, i.e., those drivers traveling more than 10 mph above the limit, a different result is obtained. As shown in Figure 6, the number of drivers exceeding the posted limit by more than 10 mph at $40-\mathrm{mph}$ locations during the night hours is 4 percent higher than the noncompliance rate for the daytime hours. A similar pattern emerged for the 25 - and 45 mph sites. Thus, excessive speeding appears more prevalent at night.

## Accident Risk

## General

Accident data were gathered for each of the roadway segments where speed data were collected. Three years of data

TABLE 7 PERCENTAGE OF DRIVERS EXCEEDING POSTED SPEED LIMIT (OVERALL, AND FOR 5, 10, AND 15 mph OVER LIMIT)

| Speed Limit | Road Type | >SL | $>5$ | >10 | $>15$ |
| :---: | :---: | :---: | :---: | :---: | :---: |
| 25 | Two-lane | 81.5 | 50.8 | 22.0 | 6.3 |
|  | Multilane | 91.0 | 70.0 | 37.0 | 11.0 |
| 30 | Two-lane | 81.7 | 62.7 | 44.0 | 23.6 |
|  | Multilane | 58.0 | 15.0 | 1.0 | 0 |
| 35 | Two-lane | 70.5 | 40.7 | 16.8 | 5.2 |
|  | Multilane | 72.4 | 46.8 | 23.6 | 8.5 |
| 40 | Two-lane | 66.0 | 34.3 | 12.0 | 3.3 |
|  | Multilane | 56.6 | 24.3 | 7.4 | 2.6 |
| 45 | Two-lane | 73.9 | 40.6 | 12.6 | 2.6 |
|  | Multilane | 74.2 | 41.0 | 14.7 | 3.4 |
| 50 | Two-lane | 58.0 | 31.5 | 10.8 | 2.3 |
|  | Multilane | 68.0 | 35.8 | 10.6 | 2.8 |
| 55 | Two-lane | 57.5 | 35.8 | 12.5 | 3.5 |
|  | Multilane | 73.0 | 42.0 | 14.0 | 3.5 |

TABLE 8 PERCENTAGE OF DRIVERS EXCEEDING POSTED SPEED LIMIT (OVERALL, AND FOR 5, 10, AND 15 mph OVER LIMIT) BY TYPE OF VEHICLE

| Speed Limit | Vehicle Type | >SL | $>5$ | >10 | $>15$ |
| :---: | :---: | :---: | :---: | :---: | :---: |
| 25 | Car | 83.7 | 54.9 | 25.2 | 7.5 |
|  | Truck | 70.0 | 38.4 | 17.8 | 3.4 |
| 30 | Car | 75.9 | 51.2 | 33.7 | 18.2 |
|  | Truck | 62.5 | 36.5 | 19.8 | 5.5 |
| 35 | Car | 71.5 | 43.8 | 19.9 | 6.6 |
|  | Truck | 57.1 | 32.1 | 13.6 | 5.2 |
| 40 | Car | 63.2 | 31.0 | 10.4 | 2.6 |
|  | Truck | 40.6 | 17.8 | 4.6 | 0.2 |
| 45 | Car | 74.9 | 41.7 | 14.8 | 3.1 |
|  | Truck | 49.4 | 21.4 | 6.3 | 0.9 |
| 50 | Car | 63.4 | 34.1 | 10.8 | 2.5 |
|  | Truck | 44.4 | 19.0 | 5.5 | 1.0 |
| 55 | Car | 63.3 | 37.1 | 13.4 | 4.5 |
|  | Truck | 52.5 | 29.8 | 12.8 | 3.0 |



FIGURE 5 Percentage of noncompliance versus speed limit for day, night, and dawn or dusk conditions.


FIGURE 6 Percentage exceeding posted speed by more than 10 mph for day, night, and dawn or dusk conditions.
were used to develop relationships between speed, accident involvement, and accident severity. However, the small number of sites limited the analysis. Of the 44 non- $55-\mathrm{mph}$ sites, 25 sites had less than 30 accidents during the 3 -year period and 10 sites had 10 or fewer accidents. On the other end of the spectrum, 5 sites had a total of 867 accidents ( 42 percent of the total 2,054 accidents in the data base) during the 3 -year period. The lower and upper extremes of the data base were 0 and 291 accidents, respectively.

## Accident Rate Analysis

For each speed limit class, calculated accident rates included overall, injury, fatal, speeding, day, and night rates. Results presented in Table 9 indicate that the rates for the $25-\mathrm{mph}$ zone were consistently higher than for any of the other zones. This finding is primarily because of the one $25-\mathrm{mph}$, multilane site that had an accident frequency of 291 in a section that was only 0.70 mi long. Examining the remaining numbers in Table 9, the lowest rates are in the $45-$ and $50-\mathrm{mph}$ zones. The highest rates, excluding the $25-\mathrm{mph}$ zone, are in the $30-$ and $55-\mathrm{mph}$ zones. Injury rates for each speed zone ranged from 28 to 50 percent of the overall rate, whereas the fatality rate was nonexistent with the exception of the 30 - and 55mph zones. The speeding accident rate was inconsistent among cells and ranged from 7 to 39 percent of the overall accident rate. Final rates, day versus night, indicated that the night rate was consistently lower than the daytime rate. This finding is in contrast to the national trend.

The next step in the accident analysis was to determine the statistical significance for the accident rates. Because the results from Table 9 indicate higher rates for the 25 - and $30-\mathrm{mph}$
speed zones, a weighted regression analysis used to test for statistical significance was computed using dummy variables for three speed zone categories- $25 \mathrm{mph}, 30 \mathrm{mph}$, and all others. The analysis was used to compare the rates for 25 and $30-\mathrm{mph}$ speed zones to the rates for all other speed zones combined. The results, presented in Table 10, indicate that all rates were significantly different at a confidence level of 95 percent with the exception of the fatal accident rate in the $25-\mathrm{mph}$ zone and the night accident rate and speeding accident rate in the $30-\mathrm{mph}$ zone.
The next step in the analysis was determining variables that may be associated with differing accident rates. Among the variables examined was driveways per mile. Using weighted regressions, accident rates were compared for those sites having fewer than 20 driveways per mile with those sites having 20 or more driveways. The results, presented in Table 11, indicate that there was a significant difference at the 95 percent confidence level for all rates except the fatal and speeding accident rates. There was no significant difference for sites with commercial development versus sites without commercial development.

## Accident Involvement Versus Speed

Prior research has indicated a relationship between accident involvement and deviation from the mean speed of the traffic stream $(5,6)$. Figure 7 shows these findings for rural highways and freeways, with the lowest involvement rate occurring at 7 and 12 mph over the mean speed, respectively. As a driver deviates from these low points, the accident risk increases.
Results of this study produced similar curves for involvement rate using non-55-mph data from North Carolina and

TABLE 9 ACCIDENT RATES—OVERALL, FATAL, INJURY, SPEEDING, DAY, AND NIGHT

| Speed <br> Limit | Overall | Fatal | Injury | Speeding | Day | Night |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: |
| 25 | 13.53 | - | 4.45 | 5.24 | 6.71 | 2.68 |
| 30 | 10.81 | 0.19 | 2.90 | 0.72 | 6.47 | 1.01 |
| 35 | 2.89 | 0.02 | 1.07 | 0.63 | 1.97 | 0.32 |
| 40 | 1.96 | - | 0.82 | 0.36 | 0.92 | 0.42 |
| 45 | 1.52 | 0.02 | 0.66 | 0.49 | 1.07 | 0.30 |
| 50 | 1.74 | 0.02 | 0.89 | 0.17 | 0.76 | 0.25 |

TABLE 10 STATISTICAL SIGNIFICANCE FOR ACCIDENT RATES

| Type of <br> Accident Rate | Speed Zone |  |  |
| :---: | :---: | :---: | :---: |
|  | 25 mph | 30 mph | All Other |
| Overall | $13.53 *$ | $10.81 *$ | 1.83 |
| Injury | $4.45 *$ | $2.90 *$ | 0.82 |
| Fatal | - | $0.19 *$ | 0.02 |
| Daytime | $6.71 *$ | $6.47 *$ | 1.19 |
| Nighttime | $2.68 *$ | 1.01 | 0.27 |
| Speeding | $5.24 *$ | 0.72 | 0.46 |

* Indicates significant difference from the All Other group at the 95 percent confidence level.

TABLE 11 ACCIDENT RATES USING WEIGHTED REGRESSIONS

| Type of <br> Accident Rate | Number of Driveways per Mile |  |
| :---: | :--- | :---: |
|  | $<20$ | 20 or more |
| Overall | 1.46 | $3.11 *$ |
| Injury | 0.64 | $1.46 *$ |
| Fatal | 0.00 | 0.03 |
| Daytime | 0.91 | $2.47 *$ |
| Nighttime | 0.27 | $0.64 *$ |
| Speeding | 0.46 | 0.91 |

* Indicates significant difference from the $<20$ group at the 95 percent confidence level.


FIGURE 7 Accident involvement and deviation from mean speed of traffic stream for rural highways and freeways $(5,6)$.

Colorado. Variation from the mean speed was plotted against involvement rate, which was defined as weekday, nonalcohol, and nonintersection involvements per 100,000 veh-mi (see Table 12 and Figure 8). The lowest involvement rate for this curve occurred at 7 mph above a mean speed of 44.2 mph at 24 involvements per 100,000 veh-mi. A closeup of this low point is shown in Figure 9. A proportion of the cumulative speed distribution curve for North Carolina and Colorado data, with respect to the variation from the mean speed, is also shown in Figure 9. On the basis of these data, the speed at which the accident risk was minimized occurred at the 90th percentile of the travel speeds observed, as shown in Figure 9 by the dashed line that projects upward from the low point of the accident involvement rate curve to the intersection of the cumulative speed distribution curve and then horizontally to the right-hand scale.

## ASSESSMENT OF CRITERIA

Criteria used to establish speed limits are important as to whether speed limits are deemed reasonable by the public and whether accident risk is truly minimized. Of the 44 non-$55-\mathrm{mph}$ sites used in this study, 21 had speed limits set on the basis of engineering studies, often with 85th-percentile speed as the governing factor. Of the remaining sites, 10 were statutory limits, 2 were based solely on engineering judgment, and the criteria by which 11 were set were unknown. In Table 13 , the percentage of vehicles exceeding the speed limit is shown for each speed limit class by the criteria used to establish the limit. In no case is compliance good, but it is extremely poor for the lower-speed zones in which statutory limits are imposed or in which engineering judgment by itself was used in setting the speed limit.
Examining the distribution of percent noncompliance, a natural breakpoint was found at 60 percent. A total of 12 sites exhibited a noncompliance rate of 60 percent or less as presented in Table 14. Also listed are the accident rates for each of those segments. The average accident rate for the 44 non-55-mph sites was 4.27 accidents per million veh-mi (MVM). Of the 12 sites with low compliance, 7 exhibited an accident rate lower than the average; of those 7,3 had speeds established on the basis of an engineering study, 3 had statutory limits, and 1 was unknown.
The speed statistics reveal that only 7 of the 44 sites had mean speeds lower than the posted speed limits, and no site had an 85th-percentile speed less than the posted limit.

## SUMMARY OF FINDINGS

Analysis of travel speed, compliance, and accident risk produced the following significant findings:

- Mean speeds exceeded the posted speed limit by 1 to 8 mph;
- 85th-percentile speeds ranged from 6 to 14 mph over the posted speed limit or 4 to 7 mph over the mean speed;
- Cars travel 1 to 5 mph faster than trucks for all speed zones;
- No consistent increase or decrease in speeds based on time of day was observed across all speed classes;
- The majority ( 70.2 percent) of free-flow drivers observed did not comply with posted speed limits;

TABLE 12 VARIATION FROM MEAN SPEED AND INVOLVEMENT RATE

| Deviation from <br> Mean Speed <br> $(\mathrm{mi} / \mathrm{h})$ | Involvements | Vehicle <br> Miles | Involvement <br> Rate* |
| :--- | :---: | :---: | ---: |
| -25.0 to -29.9 | 38 | 1486.83 | 2556 |
| -20.0 to -24.9 | 33 | 1678.41 | 1966 |
| -15.0 to -19.9 | 54 | 4518.67 | 1195 |
| -10.0 to -14.9 | 71 | 15818.39 | 449 |
| -5.0 to -9.9 | 154 | 53957.38 | 285 |
| 0.0 to -4.9 | 94 | 136799.50 | 69 |
| +0.1 to +4.9 | 63 | 141032.60 | 45 |
| +5.0 to +9.9 | 14 | 57385.67 | 24 |
| +10.0 to +14.9 | 4 | 7861.62 | 51 |
| +15.0 to +19.9 | 2 | 412.48 | 485 |
| +20.0 to +24.9 | 4 | 64.19 | 6232 |
| +25.06 to +29.9 | 1 | 14.14 | 7072 |

* Involvement rate $=$ number of involvements per 100,000 vehicle-miles.


FIGURE 8 Accident involvement rate using non- $55-\mathrm{mph}$ data from North Carolina and Colorado (weekday, nonalcohol, and nonintersection involvements per $\mathbf{1 0 0 , 0 0 0}$ veh-mi).


- NON-55 mph HWYS - PERCENTILES

FIGURE 9 Low-point accident involvement rate and cumulative speed distribution curves for North Carolina and Colorado.

TABLE 13 PERCENT OF VEHICLES EXCEEDING SPEED LIMIT FOR EACH SPEED LIMIT CRITERION

| Speed <br> Limit | Engineering <br> Study | Engineering <br> Judgement | Statutory | Unknown |
| :---: | :---: | :---: | :---: | :---: |
|  | 74.0 | --- | 95.0 | 87.0 |
|  | 77.5 | 96.0 | --- | 52.0 |
| 30 | 63.9 | --- | 90.2 | 52.5 |
| 35 | 60.1 | 76.0 | 54.9 | --- |
| 40 | 76.7 | --- | 73.0 | 64.1 |
| 45 | 74.0 | --- | 53.5 | 67.4 |

TABLE 14 STUDY SITES WITH NONCOMPLIANCE RATES OF 60 PERCENT OR LESS

| Site No. | Noncompliance (\%) | Accident Rate (Acc/MVM) |
| :---: | :---: | :---: |
| 340 | 51.0 | 0.79 |
| 2965 | 52.0 | 1.37 |
| 361 | 32.0 | 3.20 |
| 217 | 54.9 | 0.39 |
| 8170 | 58.2 | 1.71 |
| 214 | 41.6 | 5.77 |
| 133 | 58.0 | 1.47 |
| 304 | 41.0 | 0.42 |
| 436 | 58.0 | 8.76 |
| 041 | 52.0 | 19.14 |
| 047 | 58.0 | 8.31 |
| 120 | 57.0 | 8.60 |

- Overall, the percentage of drivers exceeding posted speed limits by more than 5 mph was 40.8 percent, by more than $10 \mathrm{mph}, 16.8$ percent, and by more than $15 \mathrm{mph}, 5.4$ percent;
- In general, 85 percent compliance was achieved at speeds 10 mph over the posted speed limit;
- Data indicated some reluctance by drivers to speed excessively on higher-speed roadways;
- Noncompliance was higher for cars than for trucks at all levels;
- Excessive speeding (more than 10 mph over the posted speed limit) is more prevalent at night;
- Accident rates for the $25-\mathrm{mph}$ zones were consistently higher than for any of the other zones;
- In contrast to the national trend, the night accident rate was consistently lower than the daytime rate; and
- Speed at which accident risk was minimized occurred at the 90 th percentile of the travel speeds observed.


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# Effect of the $65-\mathrm{mph}$ Speed Limit on Speeds in Three States 

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#### Abstract

Following the April 1987 enactment of federal law permitting 65mph speed limits on rural Interstate highways, 40 states adopted higher speed limits by the middle of 1988. Nondetectable radar was used to measure speeds in three states to evaluate the effect of the $65-\mathrm{mph}$ speed limit on speeds of free-flowing vehicles on Interstate highways during daytime off-peak periods (9:00 a.m. to 4:00 p.m.). In New Mexico, rural and urban speeds were measured at 2 -month intervals over a 2 -year period after the speed limit was increased in April 1987. In Virginia and Maryland, rural speed data were collected immediately before and after Virginia implemented the $65-\mathrm{mph}$ limit in July 1988 and data collection was repeated at 3 -month intervals for 1 year. Two weeks after the $65-\mathrm{mph}$ speed limit began in Virginia, mean and 85 th-percentile speeds of cars were higher by almost 3 mph , whereas the speed of tractor-trailers (still limited to 55 mph ) was unchanged. The proportion of cars exceeding 70 mph nearly doubled. Speeds of cars and trucks in neighboring Maryland (with $55-\mathrm{mph}$ speed limit) did not increase during the same 2 weeks. A longer-term trend of increasing speed was also found in Virginia. In contrast, car speeds in Maryland showed no upward trend, but tractortrailer speeds have increased to the same level as in Virginia. In New Mexico, average speeds of passenger cars and light trucks on rural highways increased nearly 3 mph within 9 months of the $65-\mathrm{mph}$ law and have since continued to increase. The proportion exceeding 70 mph grew nearly fivefold for cars and doubled for heavy trucks. Urban highway speeds in New Mexico have shown a slight net increase over 27 months, while also exhibiting pronounced seasonal variation.


When the $55-\mathrm{mph}$ national speed limit was established in 1974 as part of a broad effort to reduce energy consumption in the United States, average speeds on rural Interstate highways (as determined by each state and reported by the FHWA) dropped from 65.0 mph in 1973 to 57.6 mph (1). Since 1974, speeds both on urban and rural Interstate highways have gradually increased. Although the procedures used by states to measure, analyze, and report speeds changed between 1980 and 1982, by 1986 the average speed on rural Interstate highways increased to 59.7 mph and the 85 th-percentile speed (the speed at or below which 85 percent of the vehicles are traveling) was $66.2 \mathrm{mph}(1)$. By 1986, 76 percent of drivers exceeded 55 mph and 18 percent exceeded 65 mph on rural Interstate highways. As many as 90 percent of vehicles were traveling faster than 55 mph and more than 30 percent surpassed 65 mph in some states (1).

In April 1987, Congress enacted the Surface Transportation and Uniform Relocation Act permitting states to set a maximum $65-\mathrm{mph}$ speed limit on certain highways in the Interstate system located outside urbanized areas with populations of

[^6]50,000 or more. Further, in December 1987, Congress established a demonstration program that allowed up to 20 states to adopt a $65-\mathrm{mph}$ speed limit on three additional classes of highways outside of urbanized areas. By late 1988, 40 states had raised speed limits to above 55 mph on almost $29,800 \mathrm{mi}$ of Interstate highways and 16 states had done so on approximately $2,200 \mathrm{mi}$ of non-Interstate highways (personal communication). Nearly all of these highways are posted at 65 mph for cars, whereas 15 states restrict certain vehicles (buses, trucks, and others) to lower speeds.

As speed increases, vehicles become more difficult to control, drivers have less time to react to other vehicles and roadway hazards, stopping distances are greater, and more energy is imparted in collisions thus increasing their severity. Research has shown that rates for injury and property damage in crashes increase exponentially with precrash speed and the percentage of drivers and front-seat passengers injured increases monotonically with speed at impact $(2,3)$. Speed limits are intended to improve safety by preventing excessive speed. Reduced and more uniform travel speeds resulted in an estimated 3,000 to 5,000 fewer deaths in the year following implementation of the $55-\mathrm{mph}$ national speed limit. In addition, an estimated 4,000 fatalities were prevented in 1983 by the lower speeds, even though average speeds had increased somewhat above the 1974 level (4).

Two time series studies are presented of speeds on rural (and some urban) Interstate highways in three states: Virginia and New Mexico, which changed to a $65-\mathrm{mph}$ speed limit (although heavy trucks are still limited to 55 mph in Virginia), and Maryland, which retained the $55-\mathrm{mph}$ speed limit. The objective of the speed studies was to determine the short- and longer-term effects of the $65-\mathrm{mph}$ speed limit change on mean speed, speed distribution, and compliance for cars, light trucks, and tractor-trailers on rural Interstate highways. Virginia and Maryland data were also used to evaluate the immediate effect of the increased speed limit, and New Mexico data were used to examine urban Interstate highway speeds.

## METHODS

The New Mexico study began in April 1987 two weeks after the speed limit was raised. Speeds were measured at about 2-month intervals for 27 months until June 1989. The Virginia study was performed 2 weeks before the speed limit increase to 65 mph (July 1, 1988) and 2 weeks after the new speed limit was posted with follow-up data collection every 3 months for 1 year until July 1989. Data were collected simultaneously in adjacent Maryland, which retained the $55-\mathrm{mph}$ limit.

## Site Selection and Site Characteristics

For each state, sites were selected on the basis of geographic location, topography, roadway geometric characteristics, and observer safety (Table 1). Sites were generally located on roadway sections that had little or no gradient or curvature, except two sites in New Mexico were not closer than $1 / 2 \mathrm{mi}$ to the nearest interchange. Sites were located on each of Virginia's five rural Interstate highways and were distributed to represent the state's range of topography. In neighboring Maryland, one site on I-95 was selected to match Virginia's I-95 site, and an I-70 site was matched to the more rural nature of Virginia's sites on I-64, I-81, and I-85. Four sites in New Mexico were chosen in the vicinity of Albuquerque on I-25 and I-40, with one urban and one rural site located on each Interstate.

## Data Collection

Procedures used for data collection were similar in all three states. In each state, the speed of free-flowing vehicles was measured using nondetectable K-band radar mounted in vehicles parked either on an overpass above the travel lanes (New Mexico rural sites), off the roadway shoulder behind the guardrail, or in the clear zone (urban New Mexico and all Virginia and Maryland sites). Data collectors recorded the speed, vehicle type, travel lane, registration state, and time of day for each observation. Observer vehicles included several minivans, a small passenger car, and a small pickup truck, none of which resembled vehicles used for local and state law enforcement. Radar units had been modified by their manufacturer so that the signal could not be received by commercial radar detectors (5). Observer vehicle locations were as inconspicuous as possible, and measurements were made with radar units aimed downstream at receding vehicles. Radar calibration was checked at the beginning and end of each data collection session, which was during one weekday between 9:00 a.m. and 4:00 p.m. at each site.

In Virginia and Maryland, speeds at each site were initially measured once during each of the two successive weeks before and two successive weeks after July 1, 1988, the date on which Interstate speed limit signs were changed to 65 mph . Speeds were measured for only 1 day at each site during each sub-
sequent data collection period. Each measurement session at a particular site was performed on the same weekday (Tuesday through Thursday) throughout the study in Virginia and Maryland. Observations were made of traffic moving in one direction only. At New Mexico sites, data were collected for 90 min in one direction, then the observer moved and measured traffic speed in the opposite direction for 90 min . In Virginia and Maryland, data collectors maintained a $\log$ of police, emergency service vehicles, and other unusual activities near the sites. This information was later used to separately evaluate observations that may have been influenced by such events. At the New Mexico sites, virtually no police enforcement activities were observed during data collection.

## Sampling Procedure

Sampling was restricted to free-flowing vehicles whose headway (time separation from the previous vehicle in the same lane) was at least 5 sec . Data collectors were directed to always choose the next free-flowing vehicles in any lane following completion of a speed measurement. The protocol did not attempt to systematically sample each vehicle type according to its proportion within the overall population of all vehicles.

## Analysis Procedure

Raw data were corrected for two factors-angle of observation and radar frequency. Observation angle factor adjusts the speed upward to compensate for measured speed that decreases as a function of the cosine of the angle between the observed vehicle's path and the aim of the radar beam. Radar frequency factor accounts for the difference between speeds measured by standard radar compared with nondetectable radar, which reads 1.45 percent higher. The complete correction for observed speed is given by

Speed $=($ observed speed $)$

$$
\begin{equation*}
\div(1.0145 * \text { cosine of observation angle }) \tag{1}
\end{equation*}
$$

In order to ensure that only free-flowing, unconstrained vehicles were analyzed, observations that had been made within 2 min before and after any observed or suspected event that

TABLE 1 SPEED MEASUREMENT SITE CHARACTERISTICS

| State | Locatlon | Route | Direction | Milepost | Geographic Locatlon | Distance to nearest Interchange (miles) |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: |
| Virginia | Rural | I-64 | Westbound | 132.2 | Charlottesville | 2.0 |
|  |  | 1-66 | Westbound | 41.6 | Haymarket | 2.0 |
|  |  | 1-81 | Northbound | 163.7 | Roanoke | 1.0 |
|  |  | 1-85 | Northbound | 39.6 | Brunswick Co. | 1.4 |
|  |  | I-95 | Southbound |  |  |  |
| Maryland | Rural | 1-70 | Eastbound | 5.5 | Millstone | 0,5 |
|  |  | 1-95 | Southbound | 38.7 | Elkton | 2.3 |
| New Mexico | Rural | 1-25 | Northbound and Southbound | 248 | Algodones | at interchange |
|  |  | 1-40 | Eastbound and Westbound | 196 | Moriarity | 1.5 |
| New Mexico | Urtan | 1-25 | Northbound and Southbound | 232 | Albuquerque | 1.5 |
|  |  | 1-40 | Eastbound and Westbound | 165 | Albuquerque | at interchange |

may have influenced speed (such as presence or passage of a police vehicle, a breakdown, or citizen band radio communications that identified a radar or enforcement operation) were separated from the data base. Data from the remaining vehicles were analyzed in terms of mean speed, standard deviation, selected percentile values, and the frequency distribution of speed for each data group (6). Observations were grouped by state, observation phase, and vehicle type, and summary statistics were calculated for each group.

## RESULTS

## Immediate Effect of $\mathbf{6 5}-\mathrm{mph}$ Speed Limit

Drivers of cars and light trucks in free-flowing traffic in Virginia raised their speeds on the five rural Interstate highways immediately following implementation of the $65-\mathrm{mph}$ speed
limit. As presented in Table 2, the average speed of cars increased 2.8 mph to 65.9 mph . Tractor-trailer speeds did not increase (in fact, they decreased slightly) in Virginia because the new law still restricted them to 55 mph . Mean speeds on the two rural Interstates in Maryland did not increase but rather decreased for all vehicle types (Table 3). Speeds of cars and light trucks in Maryland after July 1 were 4.5 mph lower than those in Virginia.

In Virginia, the standard deviation (a statistical measure of the range of speed data) was virtually unchanged for cars but decreased slightly for light trucks. Coupled with the increase in mean speeds, this result suggests that the entire distribution of speeds shifted upwards for cars and light trucks because most sampled drivers simply drove faster. The 85th-percentile speed, often claimed as the basis for setting speed limits (7), increased 2.9 mph to 70.6 mph for cars. For tractor-trailers, the 85 th-percentile speed was unchanged. The mean speed of

TABLE 2 SUMMARY STATISTICS FOR VIRGINIA

| Observation Perlod | Sample Slze | Mean Speed (mph) | Std. dev. (mph) | 85th \%lle (mph) | \% exceeding 65 mph | \% exceeding 70 mph |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: |
| 55 mph Limit PASSENGER CARS |  |  |  |  |  |  |
|  |  |  |  |  |  |  |
| June 1988 | 4,784 | 63.1 | 4.9 | 67.7 | 32.1 | 8.2 |
| 65 mph LImit |  |  |  |  |  |  |
| July 1988 | 5,190 | 65.9 | 4.9 | 70.6 | 57.6 | 17.0 |
| October 1988 | 2,3171 | 65.7 | 4.9 | 70.2 | 59.2 | 17.8 |
| January 1989 | 1,589 | 67.4 | 5.1 | 72.4 | 69.8 | 30.5 |
| April 1989 | 2,201 | 66.9 | 5.0 | 71.4 | 68.7 | 26.0 |
| July 1989 | 2,020 | 66.9 | 4.8 | 71.2 | 69.3 | 25.2 |
| 55 mph LImt $\quad$ LIGHT TRUCKS* |  |  |  |  |  |  |
|  |  |  |  |  |  |  |
| June 1988 | 1,517 | 62.1 | 5.1 | 67.0 | 25.6 | 5.9 |
| 65 mph Limit |  |  |  |  |  |  |
| July 1988 | 1,711 | 65.0 | 4.8 | 69.7 | 49.8 | 13.2 |
| October 1988 | 790 | 65.0 | 5.2 | 69.4 | 53.8 | 13.8 |
| January 1989 | 537 | 66.0 | 5.3 | 71.2 | 60.5 | 22.3 |
| April 1989 | 823 | 65.5 | 5.0 | 70.2 | 58.9 | 17.4 |
| July 1989 | 741 | 66.2 | 4.7 | 70.9 | 62.1 | 21.5 |
| TRACTOR-TRAILERS |  |  |  |  |  |  |
| 55 mph Limit |  |  |  |  |  |  |
| June 1988 | 1,676 | 62.3 | 4.6 | 66.5 | 27.0 | 5.9 |
| 55 mph Limit (for Heavy Trucks; 65 mph for all others) |  |  |  |  |  |  |
| July 1988 | 1,555 | 61.8 | 4.7 | 66.5 | 21.9 | 5.5 |
| October 1988 | 1,382 | 61.4 | 4.5 | 66.1 | 20.6 | 4.0 |
| January 1989 | 995 | 61.8 | 4.4 | 66.3 | 21.1 | 4.5 |
| April 1989 | 1,475 | 61.0 | 4.4 | 65.3 | 17.8 | 2.8 |
| July 1989 | 1,017 | 61.7 | 4.5 | 66.1 | 22.4 | 5.1 |

[^7]TABLE 3 SUMMARY STATISTICS FOR MARYLAND

| Phase | Sample Slze | Mean Speed (mph) | Std. dev. (mph) | 85th \%lle (mph) | \% exceeding 65 mph | \% exceeding 70 mph |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: |
|  | PASSENGER CARS |  |  |  |  |  |
| 55 mph Limit |  |  |  |  |  |  |
| June 1988 | 2,047 | 61.7 | 5.1 | 66.5 | 23.5 | 6.6 |
| 65 mph Limit (VIrginia) |  |  |  |  |  |  |
| July 1988 | 2,525 | 61.4 | 5.2 | 66.5 | 23.5 | 5.7 |
| October 1988 | 992 | 61.1 | 5.0 | 66.4 | 21.4 | 4.6 |
| January 1989 | 559 | 63.4 | 5.6 | 68.5 | 39.3 | 10.7 |
| April 1989 | 578 | 61.9 | 5.1 | 67.3 | 28.4 | 6.1 |
| July 1989 | 575 | 61.6 | 5.4 | 67.1 | 27.8 | 6.3 |
| 55 mph Lirilt $\quad$ LIGHT TRUCKS* |  |  |  |  |  |  |
|  |  |  |  |  |  |  |  |  |  |  |  |  |
| June 1988 | 551 | 61.2 | 5.4 | 66.5 | 22.7 | 6.7 |
| 65 mph Limit (Virginia) |  |  |  |  |  |  |
| July 1988 | 599 | 60.5 | 4.8 | 65.4 | 18.7 | 3.2 |
| October 1988 | 281 | 60.2 | 5.3 | 66.3 | 20.3 | 6.0 |
| January 1989 | 204 | 61.7 | 5.2 | 66.6 | 27.9 | 6.4 |
| April 1989 | 195 | 61.3 | 4.8 | 66.3 | 24.1 | 4.6 |
| July 1989 | 187 | 61.2 | 4.8 | 66.2 | 22.5 | 3.7 |
| 55 mph Limlt $\quad$ TRACTOR-TRAILERS |  |  |  |  |  |  |
|  |  |  |  |  |  |  |  |  |  |  |  |  |
| June 1988 | 560 | 59.8 | 4.6 | 64.3 | 12.5 | 2.1 |
| 55 mph Limit (65 mph In Virginia for cars, light trucks and buses) |  |  |  |  |  |  |
| July 1988 | 663 | 59.3 | 4.4 | 63.3 | 10.0 | 1.1 |
| October 1988 | 308 | 59.6 | 4.6 | 64.3 | 14.0 | 1.9 |
| January 1989 | 631 | 61.4 | 4.4 | 66.3 | 18.7 | 4.4 |
| April 1989 | 583 | 61.4 | 4.3 | 65.5 | 19.7 | 2.2 |
| July 1989 | 566 | 61.1 | 4.5 | 66.1 | 22.3 | 2.3 |

*Light trucks defined as pickups, utility vehicles, vans, and other trucks not exceeding $\mathbf{1 0 , 0 0 0} \mathrm{lbs}$ gross vehicle weight.
cars in Maryland was slightly lower, whereas the standard deviation and 85th-percentile speed remained unchanged after July 1. These measures for light trucks and tractor-trailers in Maryland were moderately lower after July 1.

The reported standard deviations do not represent speed variance because this term has been associated with the likelihood of a crash. Rather, these standard deviations are withingroup measures combined across both free-flowing and constrained vehicles and across all vehicle types. The sampling and data collection methods do not allow estimation of the standard deviation of the population of all vehicles.
The issue of the immediate effect of compliance with the law can be examined through analysis of the mean speed and the percentage exceeding the speed limit. Following the change to a $65-\mathrm{mph}$ speed limit, the mean speed of cars and light trucks in Virginia became more closely aligned with the speed limit. However, the proportions of free-flowing vehicles
exceeding 65 and 70 mph each doubled (Table 2). The proportion of vehicles exceeding 65 and 70 mph in Maryland during this time was unchanged for cars and decreased for other vehicles.

## Longer-Term Effects of the $\mathbf{6 5}$-mph Speed Limit

## Virginia and Maryland

During the first full year following the speed limit increase in Virginia, the mean speed of cars and light trucks on rural Interstate highways increased an additional 1 mph above the initial increase, as shown in Figure 1. Speeds increased 2 to 3 mph by October 1988 and were highest in January 1989 (up 4 mph from their levels just before the speed limit increase),


FIGURE 1 Mean and 85th-percentile speeds on Virginia and Maryland rural Interstates.
but they subsequently decreased slightly for cars. In contrast, neighboring Maryland's car and light truck speeds were slightly lower during the 4th month after Virginia's speed limit increased. Longer-term trends of the percentages of cars and light trucks exceeding 65 and 70 mph paralleled the speed trends in Maryland and Virginia. The proportions of these vehicles exceeding 65 and 70 mph both doubled initially, whereas Virginia experienced a continuing upward trend
(Figure 2). For cars, the proportion exceeding 65 mph increased to a peak of 69.8 percent in January 1989 and remained at nearly 70 percent through April and July 1989. The proportion exceeding 70 mph also increased to a January peak of 30.5 percent, which subsequently leveled off to about 25 percent. Maryland drivers demonstrated no initial increases in the proportion exceeding 65 and 70 mph between June and July 1988. Peak values were observed in January 1989 and by April

free-flow vehicies; non detectable radar
FIGURE 2 Proportion of vehicles exceeding 65 and 70 mph on Virginia and Maryland Interstates.
returned to levels similar to those before Virginia's speed limit increased.

For tractor-trailers, little change occurred during the year in either speed distribution or percentage exceeding 65 and 70 mph in Virginia, whereas in Maryland both showed an increased trend. By July 1989, distributions of tractor-trailer speeds in Virginia and Maryland were nearly identical.

## New Mexico

In New Mexico, the longer-term trend ( 27 months) of freeflowing car and truck speeds on rural Interstate highways posted at 65 mph was gradually upward, with annual peaks evident each December (Table 4). As shown in Figure 3, speeds for cars, light trucks, and tractor-trailers increased

TABLE 4 SUMMARY STATISTICS FOR NEW MEXICO RURAL INTERSTATE SITES FOLLOWING IMPLEMENTATION OF 65-mph SPEED LIMIT

| Phase | $\begin{aligned} & \text { Sample } \\ & \text { SIze } \end{aligned}$ | Mean Speed (mph) | Std. dev. (mph) | 85th \%lle (mph) | \% exceeding 65 mph | $\begin{aligned} & \text { \% exceeding } \\ & 70 \mathrm{mph} \end{aligned}$ |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: |
| PASSENGER CARS |  |  |  |  |  |  |
| April 1987 | 766 | 63.5 | 4.3 | 67.3 | 36.8 | 5.1 |
| June 1987 | 718 | 64.8 | 4.6 | 69.3 | 49.4 | 12.3 |
| August 1987 | 760 | 64.3 | 4.4 | 68.3 | 45.4 | 10.3 |
| November 1987 | 777 | 66.1 | 5.0 | 71.2 | 60.9 | 21.0 |
| December 1987 | 887 | 66.8 | 5.3 | 71.2 | 64.8 | 22.8 |
| February 1988 | 776 | 66.0 | 4.9 | 71.2 | 59.1 | 19.6 |
| April 1988 | 824 | 66.4 | 5.5 | 71.2 | 62.0 | 22.2 |
| June 1988 | 817 | 66.4 | 4.8 | 71.2 | 61.9 | 21.3 |
| August 1988 | 843 | 66.9 | 5.3 | 71.2 | 66.3 | 23.4 |
| October 1988 | 791 | 66.9 | 5.0 | 71.2 | 66.6 | 25.7 |
| December 1988 | 794 | 67.8 | 5.4 | 72.2 | 72.3 | 29.3 |
| February 1989 | 842 | 66.9 | 5.2 | 71.2 | 65.3 | 23.3 |
| April 1989 | 827 | 66.8 | 5.2 | 71.2 | 64.8 | 22.9 |
| June 1989 | 774 | 66.7 | 4.9 | 71.2 | 66.1 | 23.6 |
| LIGHT TRUCKS* |  |  |  |  |  |  |
| April 1987 | 661 | 63.0 | 5.8 | 68.3 | 40.1 | 8.6 |
| June 1987 | 484 | 64.2 | 5.3 | 69.3 | 46.9 | 13.4 |
| August 1987 | 599 | 64.2 | 5.2 | 69.3 | 43.4 | 12.2 |
| November 1987 | 579 | 64.8 | 5.6 | 70.3 | 49.4 | 16.6 |
| December 1987 | 696 | 66.0 | 5.3 | 71.2 | 58.6 | 21.7 |
| February 1988 | 654 | 65.3 | 4.9 | 70.3 | 57.2 | 17.9 |
| April 1988 | 610 | 65.2 | 6.1 | 70.3 | 52.3 | 18.2 |
| June 1988 | 629 | 65.2 | 5.9 | 71.2 | 51.5 | 19.6 |
| August 1988 | 605 | 65.5 | 5.6 | 70.3 | 55.5 | 19.8 |
| October 1988 | 572 | 65.9 | 5.5 | 71.2 | 56.1 | 21.7 |
| December 1988 | 622 | 66.7 | 5.3 | 72.2 | 53.7 | 24.4 |
| February 1989 | 577 | 66.1 | 5.4 | 71.2 | 58.2 | 21.8 |
| April 1989 | 609 | 65.4 | 5.7 | 70.3 | 56.3 | 19.7 |
| June 1989 | 638 | 65.7 | 5.8 | 71.2 | 56.4 | 21.6 |
| TRACTOR-TRAILERS |  |  |  |  |  |  |
| April 1987 | 445 | 62.9 | 4.8 | 67.3 | 33.7 | 7.2 |
| June 1987 | 560 | 63.9 | 4.9 | 68.3 | 41.6 | 10.7 |
| August 1987 | 485 | 63.7 | 4.9 | 69.3 | 40.8 | 10.7 |
| November 1987 | 511 | 64.5 | 5.3 | 69.3 | 46.6 | 13.7 |
| December 1987 | 371 | 64.0 | 5.2 | 69.3 | 42.6 | 12.7 |
| February 1988 | 496 | 63.9 | 5.1 | 69.3 | 42.1 | 11.7 |
| April 1988 | 467 | 64.2 | 4.8 | 69.3 | 43.9 | 10.9 |
| June 1988 | 449 | 64.6 | 5.0 | 69.3 | 47.7 | 14.5 |
| August 1988 | 499 | 64.3 | 5.1 | 69.3 | 43.7 | 14.0 |
| October 1988 | 549 | 64.9 | 5.3 | 70.3 | 51.9 | 16.9 |
| December 1988 | 533 | 64.7 | 5.2 | 69.3 | 48.0 | 14.1 |
| February 1989 | 537 | 64.0 | 4.8 | 68.3 | 40.2 | 9.1 |
| April 1989 | 502 | 64.6 | 4.8 | 69.3 | 48.2 | 13.3 |
| June 1989 | 536 | 64.7 | 5.1 | 69.3 | 48.5 | 14.0 |

${ }^{*}$ Light trucks defined as pickups, utility vehicles, vans, and other trucks not exceeding $10,000 \mathrm{lbs}$. gross vehicle weight.
during the first 9 months of the $65-\mathrm{mph}$ speed limit but then increased at a slower pace through the last data collection period in June 1989. The mean speed of cars increased approximately 3 mph between April and December 1987 and stabilized at about 67 mph except for winter peaks. The 85thpercentile speeds of cars followed a more accelerated trend, increasing 2 mph in the first 2 months to 69.3 mph , then increasing another 2 mph to 71.2 mph by November 1987, where it remained. Tractor-trailer speeds increased a total of about 2 mph for mean and 85th-percentile speeds. The standard deviation of speed for cars and light trucks varied considerably from 4.3 to 6.1 mph , whereas for trucks a smaller range was observed. However, no apparent correlation existed between speed and standard deviation within each vehicle type.

The proportion of vehicles exceeding 65 and 70 mph on New Mexico rural Interstate highways increased more sharply than did mean and 85 th-percentile speeds (Figure 3). For cars, the proportion exceeding 65 mph doubled over the 27 -month pcriod, first sharply increasing to 64.8 percent by December 1987 and eventually reaching the 65 to 66 percent level of the last 6 months. The proportion exceeding 70 mph increased almost fivefold.

## Urban Interstate Speeds

New Mexico's urban Interstate mean and 85th-percentile speeds increased only slightly, as shown in Figure 4. The mean speed


FIGURE 3 Speeds and proportion of vehicles exceeding 65 and 70 mph on New Mexico rural Interstates.

free-flow vehlcles; non-detectable radar
FIGURE 4 Speeds and proportion of vehicles exceeding 65 and 70 mph on New Mexico urban Interstates.
of cars increased just 0.8 mph and the 85th-percentile speed of cars increased 1.0 mph by June 1989. Tractor-trailer speeds also showed little change during the 27 -month period (Table 5).

The proportion of vehicles exceeding 65 and 70 mph on urban Interstates increased slightly in New Mexico over the study period. Approximately one-third of cars exceeded 65 mph with little change over 27 months. Similarly, slightly more than 1 in 10 cars exceeded 70 mph throughout the study period. The proportion of tractor-trailers traveling in excess of 65 mph fluctuated between a high of almost 30 percent in June 1987 and a low of about 15 percent in February 1989, but was generally about 20 percent. The proportion of those exceeding 70 mph fluctuated between 2.1 and 7.5 percent, but was most often in the range of 3 to 5 percent.

## DISCUSSION

The analysis indicates that the change to a $65-\mathrm{mph}$ speed limit on rural Interstate highways in Virginia and New Mexico was associated with substantial increases in the speeds of vehicles permitted to travel at 65 mph .

Speeds were measured in Maryland and Virginia just before and after the speed limit was raised in Virginia, so the speed increases that occurred in Virginia were clearly related to the change in the speed limit. Furthermore, the speeds of tractortrailers in Virginia, where a speed limit of 55 mph was retained, did not increase.

Both in Virginia and Maryland, tractor-trailers are limited to 55 mph . The mean speed of tractor-trailers increased in

TABLE 5 SUMMARY STATISTICS FOR NEW MEXICO URBAN INTERSTATE SITES FOLLOWING IMPLEMENTATION OF 65 -mph SPEED LIMIT

| Phase | Sample Slze | Mean Speed (mph) | Std. dev. (mph) | 85th \%ile (mph) | \% exceeding 65 mph | \% exceeding 70 mph |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: |
| PASSENGER CARS |  |  |  |  |  |  |
| April 1987 | 1,013 | 62.7 | 6.2 | 69.1 | 34.1 | 11.8 |
| June 1987 | 1,064 | 63.4 | 6.0 | 69.2 | 37.1 | 12.8 |
| August 1987 | 1,100 | 63.5 | 6.0 | 69.2 | 36.9 | 14.0 |
| November 1987 | 1,115 | 64.3 | 6.0 | 70.2 | 42.7 | 17.0 |
| December 1987 | 1,056 | 64.4 | 6.0 | 71.1 | 41.4 | 17.6 |
| February 1988 | 1,125 | 64.5 | 6.2 | 71.1 | 44.4 | 18.5 |
| April 1988 | 1,057 | 64.3 | 6.1 | 70.2 | 41.4 | 17.1 |
| June 1988 | 1,092 | 62.9 | 5.4 | 68.2 | 34.0 | 10.9 |
| August 1988 | 1,136 | 63.1 | 5.7 | 69.1 | 35.5 | 12.9 |
| October 1988 | 1,050 | 63.2 | 5.5 | 68.3 | 36.0 | 11.2 |
| December 1988 | 1,046 | 63.5 | 5.5 | 69.1 | 37.7 | 12.6 |
| February 1989 | 1,031 | 63.3 | 6.0 | 69.2 | 35.2 | 13.1 |
| April 1989 | 1,063 | 63.9 | 6.3 | 70.2 | 40.2 | 16.6 |
| June 1989 | 1,086 | 63.5 | 5.7 | 70.1 | 38.0 | 15.2 |
| LIGHT TRUCKS* |  |  |  |  |  |  |
| April 1987 | 611 | 62.2 | 5.9 | 68.2 | 30.1 | 10.6 |
| June 1987 | 548 | 62.5 | 5.8 | 68.2 | 30.3 | 9.9 |
| August 1987 | 575 | 62.9 | 6.0 | 68.2 | 33.6 | 10.6 |
| November 1987 | 650 | 64.1 | 5.8 | 69.3 | 40.5 | 14.2 |
| December 1987 | 669 | 63.5 | 5.4 | 69.1 | 40.5 | 12.0 |
| February 1988 | 569 | 63.6 | 5.5 | 69.2 | 37.8 | 14.2 |
| April 1988 | 624 | 64.1 | 5.6 | 70.2 | 42.9 | 17.1 |
| June 1988 | 633 | 63.1 | 5.6 | 68.3 | 36.8 | 11.4 |
| August 1988 | 561 | 62.9 | 5.3 | 68.2 | 34.2 | 9.6 |
| October 1988 | 579 | 63.4 | 5.9 | 69.2 | 37.5 | 13.6 |
| December 1988 | 632 | 63.0 | 5.8 | 68.3 | 33.1 | 12.0 |
| February 1989 | 636 | 63.3 | 5.6 | 69.3 | 36.8 | 14.8 |
| April 1989 | 632 | 63.7 | 5.9 | 69.2 | 40.3 | 13.1 |
| June 1989 | 610 | 63.1 | 5.4 | 68.3 | 35.6 | 12.1 |
| TRACTOR-TRAILERS |  |  |  |  |  |  |
| April 1987 | 321 | 61.0 | 4.6 | 65.9 | 19.0 | 2.5 |
| June 1987 | 318 | 62.0 | 5.3 | 67.3 | 29.6 | 7.5 |
| August 1987 | 271 | 61.4 | 5.1 | 66.3 | 26.2 | 3.7 |
| Novernber 1987 | 215 | 61.8 | 4.5 | 66.2 | 20.9 | 6.0 |
| December 1987 | 247 | 61.2 | 4.0 | 65.3 | 18.6 | 2.8 |
| February 1988 | 260 | 61.2 | 4.4 | 65.2 | 17.7 | 5.0 |
| April 1988 | 292 | 61.6 | 4.2 | 65.3 | 19.5 | 4.5 |
| June 1988 | 254 | 60.9 | 4.4 | 65.3 | 20.1 | 3.5 |
| August 1988 | 286 | 60.8 | 4.4 | 65.2 | 17.8 | 2.1 |
| October 1988 | 334 | 61.7 | 4.5 | 66.3 | 24.0 | 4.5 |
| December 1988 | 294 | 60.7 | 4.2 | 65.2 | 16.7 | 3.4 |
| February 1989 | 307 | 60.4 | 4.3 | 65.2 | 15.6 | 2.3 |
| April 1989 | 272 | 60.6 | 4.2 | 65.2 | 16.5 | 2.9 |
| June 1989 | 258 | 61.3 | 4.6 | 66.1 | 21.7 | 3.9 |

[^8]Maryland to the same level (about 61 mph ) as in Virginia, where tractor-trailer speeds did not change over a 1-year period. One reason may be that a general trend of higher speed among tractor-trailer drivers is emerging because of widespread exposure to $65-\mathrm{mph}$ speed limits in many states (at least 29 states allow 65 mph for heavy trucks), a strong interest in minimizing trip time, and a desire to keep up with car traffic. Continued monitoring of speeds in these and other states may provide additional insight into the causes and consequences of this phenomenon.
The reported standard deviations are for specific vehicle types-free-flowing cars, light trucks, and tractor-trailersand do not represent estimates of the speed variance of the population of all vehicles because of intermixing in the traffic stream. However, the reported standard deviations do support conclusions regarding the range of speeds of each vehicle type sampled, but cannot be used to evaluate the interactions among vehicles.
The consequence of the $65-\mathrm{mph}$ speed limit and higher speeds has been more deaths. For the 38 states that increased speed limits, 15 to 16 percent more fatalities occurred on rural Interstate highways in 1987 than would have been expected had the $55-\mathrm{mph}$ speed limit been retained $(8,9)$. In 1988 , these same 38 states experienced 26 to 29 percent more deaths than they would have if the $55-\mathrm{mph}$ speed limit had been retained (10). The trend in speeds in Virginia and New Mexico has been gradually upward since the initial large speed increases were observed in those states. Speeds are probably increasing in other states that have adopted the $65-\mathrm{mph}$ speed limit. Consequently, the mortality consequences of higher speed limits will continue to increase.

The $65-\mathrm{mph}$ speed limit has not been found to eliminate speeders (i.e., vehicles traveling in excess of the posted speed limit). One effect of changing from the $55-\mathrm{mph}$ speed limit to the $65-\mathrm{mph}$ speed limit on rural Interstate highways has been to reduce the proportion of traffic technically in violation of the speed limit law. However, the new speed limit has greatly increased the number of high-speed vehicles. The number of automobile drivers exceeding 70 mph has increased fivefold in New Mexico and threefold in Virginia. Approximately two-thirds of the cars observed on these rural highways in Virginia and New Mexico exceeded the $65-\mathrm{mph}$ speed limit during the most recent data collection period in each state. The $65-\mathrm{mph}$ speed limit has had only limited success in recasting drivers who were speeding violators on $55-\mathrm{mph}$ roadways as law-abiding drivers. Instead, because average speeds have
increased and, as evidenced by 85th-percentile speeds, the fastest drivers are going even faster, more high-speed violators now exist than when speeds were limited to 55 mph .

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# Rural Accident Rate Variations with Traffic Volume 

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#### Abstract

The nature of the relationship between hourly traffic volumes and hourly accident rates on rural highways in New Mexico was examined. The data base consisted of traffic volumes at 44 permanent count stations and 3 years' accident experience on $10-\mathrm{mi}$ roadway sections surrounding these stations. The highest accident rates occurred during hours with the lowest traffic volumes. Over the range of conditions examined, accident rates decreased with increasing traffic volumes and with increasing volume/capacity ratios. However, because of the moderate traffic volumes on these roadways, it was not possible to determine the effect on the accident rate as hourly traffic volumes approach capacity. In rural states such as New Mexico, further study of this issue should focus on higher-volume locations found in urban areas.


For over 50 years, analysts have attempted to identify the relationship between traffic volumes and accident rates. Knowledge of this relationship, coupled with information on the capacity of the highway section, would allow a planner to estimate the safety implications of projected traffic growth and potential improvements in highway capacity. With some notable exceptions, previous research has found that the rate of traffic accidents on sections of roadway, as measured by the number of accidents per million vehicle miles (mvm) of travel, increases with increasing traffic volume. However, the data supporting this relationship are highly scattered and are sensitive to the design and operational features of the highway.

The implications of the existence of a definite relationship between traffic accident rates and the ratio of current or projected traffic volume to capacity $(\mathrm{v} / \mathrm{c})$ are quite significant. It might be expected, for example, that above some cutoff value, the rate of accidents would intensify with an increase in $\nu / c$ ratios. If this point and the nature of the relationship could be established, then a competent analyst could predict the safety implications of a growth in traffic volume or a change in highway capacity. This ability would be extremely beneficial to an agency attempting to select an optimal program of highway improvements under the constraint of a limited budget. It could also alert engineers and planners that particular roadway sections are approaching conditions that might cause roadway safety to become degraded.

As a practical matter, the relationship between accident rates and $v / c$ ratios is inherently difficult to establish. It is generally agreed that the reporting of traffic accidents is not complete, although there may be ways to address this problem (1). In addition, the quality of accident data reporting, especially with respect to crash location, is suspect. Furthermore,

[^9]the reliability of traffic volume data is certainly subject to questioning. In other words, most state data systems are not able to provide accident or volume data in a complete, accurate format, which is necessary for this type of analysis. A study was conducted to determine if the traffic records maintained in New Mexico could be used to identify a tentative relationship between accident rates and $v / c$ ratios on rural highways.

## BACKGROUND

The occurrence of a traffic accident reflects a shortcoming in one or more components of the driver-vehicle-roadway system. Typically, the correction of problems associated with one of these components is sufficient to keep an accident from occurring. Thus, although many individual factors may contribute to an accident, improvements to highways can have a significant effect in reducing crash experience. It is prudent for a highway agency to operate a program designed to identify and correct locations or situations with unusually high accident experience. Such a program is hampered, however, by the partially random nature of crash occurrence and the quality of available accident data. For this reason, there is considerable interest in surrogate measures, such as $v / c$ ratios, that may help in identifying problems.

One of the earliest studies of the relationship between traffic volumes and accident rates was reported in 1937 by Veh (2). The study examined the accident rates on sections of twolane roads in New Jersey and related these rates to the average daily traffic (ADT). Veh (2) concluded the following:

> It is interesting to note the rather definite relationship between daily traffic volume and the number of accidents per million vehicle-miles. In other words, as the average daily traffic increases, accident experience, on a million vehicle-mile basis, likewise increases to approximately 7,000 vehicles per day, which is considered by many authorities to be the capacity of a two-lane highway. Beyond that point, primarily because of congestion and the resultant decrease in speed and flexibility of movement during heavy traffic hours, there is a gradual decrease in the accident rate, despite the increase in traffic.

Although Veh (2) does not document the original data set used in the analysis, he does present a graph of the relationship between accident rates and ADT. Figure 1 shows Veh's data along with his best-fit line, which shows the relationship between the two variables. The data dispersion in Figure 1 is a reasonable representation of the amount of scatter reported subsequently by other researchers.


FIGURE 1 Relationship between accident rates and ADT (2).

Lundy (3) analyzed traffic accident experience on 659 mi of four-, six-, and eight-lane California freeways from 1960 through 1962. For each type of facility, accident rates increased with increasing ADT. In addition, Lundy observed that, for any given ADT, the four-lane freeway had the highest accident rate and the eight-lane freeway had the lowest rate. Lundy developed a set of regression equations of the following form:

Accident rate $=\alpha+\beta *$ ADT $\quad \alpha, \beta>0$
Although the values of $\alpha$ are similar for the three types of freeways, the values of $\beta$ decrease from four- to six- to eightlane facilities. A discussion of Lundy's paper by Byington (3) noted that it may be appropriate to incorporate segment length in an accident prediction model. Another discussion by Champagne (3), using Lundy's data, showed a consistent increase in accident rates as the $v / c$ ratio increased.

An NCHRP study (4) examined the accident experience on roadway sections in three states in relation to traffic volumes as well as an assortment of highway design and operational features. The study found that the single-vehicle accident rate decreased with increasing ADT, whereas the multiple-vehicle accident rate increased with increasing ADT. However, the researchers were unable to define a relationship between total accident rate and ADT because they found "in some instances the rate increased with increasing ADT and in other instances the rate decreased with increasing ADT."
A 1969 state-of-the-art review (5) examined these and other articles in the technical literature and concluded the following:

> It has been shown that accident rates tend to increase sharply with average daily traffic. Whether the levels of service which are being increasingly used to describe highway operation are related to the accident rate is unknown. Definition of this relationship might provide a useful method to predict future operational effectiveness of roadway designs.

The previously cited studies focused on the relationship of accident rates to ADT. In the mid-1960s, Gwynn (6) examined the hourly accident experience on a $3.8-\mathrm{mi}$ section of a fourlane divided highway in New Jersey. During the 5 -year study period (1959 through 1963), the ADT on this section was approximately 64,000 vehicles per day (vpd), for a total travel
of 445 mvm . Directional hourly volumes on the section ranged from 200 to 3,300 vehicles per hour (vph). Gwynn found the highest accident rates ( $4 / \mathrm{mvm}$ to $6 / \mathrm{mvm}$ ) during hours in the low-volume ranges. Rates decreased to approximately $2 / \mathrm{mvm}$ during those hours in which the directional volume was between 1,000 and $1,800 \mathrm{vph}$. At even higher hourly volumes, rates increased to $4 / \mathrm{mvm}$ to $5 / \mathrm{mvm}$. Subject to some scatter in the data, a plot of the accident rates as a function of hourly volume assumed a U shape.

Dart (7) studied 5 years of accident experience (1962 through 1966) on $1,000 \mathrm{mi}$ of rural highway in Louisiana in an attempt to determine which geometric variables contribute the most to traffic accidents. As part of this effort, he analyzed the relationship between accident rates and a traffic volume ratio, with the latter defined as the ratio of peak-hour volume to the service volume at Level of Service (LOS) B. Calculation of the traffic volume ratio required that specific design and operational features be collected for each of the 246 study sections. The ratio varied from a low of 0.04 to a high of 2.12. Dart reported that, for the traffic volume ratio range of 0.1 to 1.2 , which presumably accounted for the bulk of the study sections, the accident rate exhibited a continuous increase with increasing traffic volume ratio.

Taylor (8) described a hazardous rating formula incorporating both accident and nonaccident measures. The model included the $v / c$ ratio "because it incorporates the basic volume information, and yet normalizes these data to compensate, to some extent, for the number of lanes, traffic mix, control devices, etc." Because the model was designed to serve as a screening tool for identifying hazardous locations, Taylor minimized the data required for its application by defining the $v / c$ ratio in a nontraditional manner as ADT divided by 24 times hourly capacity. On the basis of this definition and technical input from a panel of experts, a $v / c$ ratio of more than 0.52 was judged as hazardous. As a point of comparison, the high-volume section in New Jersey studied by Gwynn would by this definition have a $v / c$ ratio of approximately 0.44 .

In 1980, Orne (9,p.20) described some preliminary efforts to examine the relationship between traffic accidents in Michigan and the actual traffic volumes at the time of the accident. This approach, which had also been suggested by Gwynn (6), is hampered to a considerable extent by the availability of reliable traffic volume data at accident sites.

More recently, researchers in Greece (10) examined the relationship between traffic accident rates and $v / c$ ratios on an 11.2-mi section of a four-lane undivided toll road. Because the facility was a toll road, with access available only at the ends of the section, it provided reliable traffic volume data. Although traffic volumes were not specifically indicated, an analysis of the number and rate of accidents for the 89 -month study period suggested that ADT values were in the range of 17,000 to $18,000 \mathrm{vpd}$. The analysis concentrated on 9.3 mi of the road, which experienced 517 accidents, for an average accident rate of approximately $1.2 / \mathrm{mvm}$. Using the techniques of the 1985 Highway Capacity Manual (HCM) (11), the researchers calculated the capacity of each of the 15 study sections and determined the $v / c$ ratio and level of service at the time of each accident. For LOS A and B operating conditions, which accounted for 81 percent of the travel and 71 percent of the accidents, the accident rate was approximately

10 percent below the average rate. At LOS C, the rate was close to the overall average. However, at $v / c$ ratios greater than 0.65 , corresponding to LOS D through F , the accident rates increased sharply. For LOS F, a forced-flow condition, the accident rate was approximately $2.9 / \mathrm{mvm}$.

The results of the Greek study are encouraging, but they must be viewed circumspectly. At the most basic level, how well the procedures of the 1985 HCM apply to the driving environment in Greece is not known. Also, the accident study conducted over the 7 -year period was subject to changes in the environment, even though there were no changes to the roadway itself. In addition, the accident sample size was relatively small in comparison with several of the other studies cited previously.

Perhaps the most extensive evaluation of this subject was a 1967 to 1975 study (12) of eight sections of four-lane interurban road in Israel. The study, which was limited to fatal and injury accidents on weekdays, contained thorough analyses both of single- and multiple-vehicle crashes. The study included two primary procedures: (a) a time-sequence analysis for each roadway section and (b) a cross-section analysis on an annual basis. Single-vehicle accident rates were extraordinarily high for flow rates below 250 vph . The multiplevehicle accident rates were more diverse, with half the sites showing a substantial increase in rates for flow rates greater than about 900 vph , and the remaining sites exhibiting little change with increases in hourly traffic volumes. When the two crash types were combined, the results were dominated by the data for multiple-vehicle crashes. More specifically, those study sections that encompassed a broad range of traffic volumes had a U-shaped relationship when accident rates were plotted as a function of hourly volume; the minimum rate occurred near 500 vph . The remaining four sites, three of which did not have hourly volumes in excess of $1,000 \mathrm{vph}$, did not show an increase in accident rates as hourly volumes increased.

The technical literature offered some useful guidance for the development of the methodology to be used in this study. It appears valid to assume that there is a relationship (albeit unknown at this time) between traffic accident rates and traffic volume. Because a variety of driver, vehicle, and roadway factors contribute to crash occurrence, it would of course be unrealistic to expect traffic volume variations to explain a large share of the differences in accident rates. The data scatter observed in these studies is a partial reflection of this problem.

The literature suggests two approaches for achieving a suitable sample for studying this issue. One procedure is to select a comparatively short section of road and to study its volume and accident characteristics for an extended period of time. By using a single, reasonably homogeneous site it is possible to thoroughly monitor the parameters of interest. On the other hand, the site chosen may not be representative of the remainder of the road system. Furthermore, accident studies that continue for an extended period are not able to account for system-wide changes in the driver, vehicle, or environment variables. An alternate approach is to use a larger number of study sections. A major advantage of this tactic is that it encompasses the characteristics of a broader section of the roadway system and offers the potential for conducting the study over a shorter period. However, it is difficult to ensure
that the sections are comparable with respect to those design and operational characteristics that can affect crash occurrence. All things considered, it appears that the use of multiple sections of constant length is the superior study procedure.

## STUDY DESIGN

The essential requirement for a study of the relationship of accident rates to $\mathrm{v} / \mathrm{c}$ ratios is reliable information on traffic accidents, hourly traffic volumes, and factors that influence highway capacity.

New Mexico maintains a computerized accident record system that includes approximately 50,000 accidents reported annually throughout the state. The system allows accidents to be selected by route and location and classified according to characteristics of interest to this study.

Traffic volume data pose a somewhat more serious problem. In New Mexico, rural traffic volumes are collected at 50 permanent count stations, and short-term counts are conducted throughout the state on a rotating basis. A recent critique by the New Mexico State Highway and Transportation Department identified valid concerns about the reliability of data from the short-term counts. Although the root problems that affected the data quality have been corrected, it will take 1 or 2 years to develop a suitable traffic volume data base for the rural state highway system. Therefore, this study relied on traffic volume data from the permanent count stations; these data are reportedly reliable.

Another principal data need is the information necessary to calculate capacity. Roadway sections at the study sites include two-lane roads, a couple of four-lane divided highways, and four-lane freeways. With a small sacrifice in accuracy, all can be assumed to have $12-\mathrm{ft}$ lanes and clear roadsides. When necessary for capacity calculation, a reasonable judgment can be made regarding driver familiarity with the roadway. A major factor affecting capacity is the combination of terrain and traffic composition, especially of heavy trucks; these parameters are available for some of the sites and could be measured at the others. For two-lane sites, the extent of the road subject to passing restrictions is necessary for the calculation of capacity; unfortunately, this information is not readily available in existing record systems.

## ASSEMBLING THE DATA BASE

In accordance with Byington's recommendation (3), the study plan involved the identification of constant-length segments around each of the permanent count stations. The length chosen for this purpose was 10 mi . A review of maps and the roadway inventory file found that six of the permanent count locations were on sections of road with significant numbers of access and egress points, to the extent that the volume counted at the permanent count station may not reflect the volume on the entire $10-\mathrm{mi}$ section; therefore, these sites were deleted from the analysis. At three additional sites, the permanent count station was on a section bounded by a pair of major intersections or interchanges less than 10 mi apart; in these cases, the sections were shortened to 5 mi . In the remaining 41 cases, it was possible to identify a $10-\mathrm{mi}$ section
around the permanent count station that was reasonably homogeneous and free of significant access points. The resulting 44 study sites, with a length of approximately 425 mi , had a combined annual travel of 960 mvm (on the basis of the FY 1988 travel volumes). The average daily traffic at individual sites ranged from 500 to $26,200 \mathrm{vpd}$.
The next step in creating the data base involved a search of the computerized accident records to identify all reported traffic accidents (fatal, injury, and property damage only) on these 44 sections for the 3-year period from 1985 to 1987. The search identified a total of 2,006 accidents, for an overall accident rate of $0.70 / \mathrm{mvm}$. The accidents were subsequently categorized by hour of the day and collision type (single- versus multiple-vehicle crashes).

## DATA ANALYSIS

In an effort to establish a tie to the results reported in the technical literature, the initial phases of the analysis examined the volume and accident experience for the entire data base. Figure 2 shows the hourly variation in total annual vehicle miles of travel for the 44 study sections. Peak travel occurs in the hours of 4:00 to 5:00 p.m. and 5:00 to 6:00 p.m., both of which account for approximately 7.4 percent of the ADT. Because of rural travel characteristics, there is no discernible morning peak but rather a gradual increase in hourly travel from about 7:00 a.m. through the late afternoon. At the other extreme, the hours beginning at 2:00, 3:00, and 4:00 a.m. each account for about 0.9 percent of the daily travel.
Figure 3 shows the hourly distribution of accidents for the 3 -year study period. The most striking feature of this graph is that it exhibits substantially less hourly variation than was evident for the traffic volumes. The maximum number of accidents (127) occurred in the hour beginning at 6:00 p.m. (6.3 percent of the daily total), followed by 5:00 p.m. and 7:00 a.m., each with about 5.6 percent. By contrast, the lowest number of accidents (54) occurred between 4:00 and 5:00 a.m., accounting for 2.7 percent of the total. When hourly accident rates are calculated, therefore, the peak rates do not occur during those hours with the greatest number of accidents but rather during those hours with the lowest volumes of traffic. As shown in Figure 4, the peak accident rate ( $3.2 / \mathrm{mvm}$ ) occurred between 2:00 and 3:00 a.m.; all the hours from 11:00 p.m. to


FIGURE 2 Hourly travel variations.


FIGURE 3 Traffic accidents, 1985 through 1987.


FIGURE 4 Traffic accident rates, 1985 through 1987.

5:00 a.m. had accident rates that were more than twice the overall daily average of $0.7 / \mathrm{mvm}$. The finding that accident rates are high during the hours of darkness is not unexpected. However, the relatively constant daytime accident rate was not anticipated. For the relatively high-volume hours between 6:00 a.m. and 7:00 p.m., only the hours beginning at 7:00 a.m. $(0.78 / \mathrm{mvm})$ and 6:00 p.m. $(0.74 / \mathrm{mvm})$ had accident rates greater than the average for the entire day.

Figure 5 shows the same information included in Figure 4 but in a manner that highlights the pattern of accident rates versus average hourly volumes. Over the range of volumes studied, the data demonstrate that rural accident rates decrease with increasing hourly volumes. Although this same pattern emerges in subsequent analyses, a cause-and-effect relationship was not assumed, especially between the low traffic volumes and their accompanying high accident rates. However, mentioning two potential explanations for this pattern is appropriate.

Single-vehicle accidents have been shown (13) to be a serious problem in New Mexico, accounting for approximately 45 percent of the state's highway fatalities. Hourly accident rates were therefore calculated separately for single- and multiple-vehicle accidents. For the entire day, the rates of single- and multiple-vehicle accidents were $0.44 / \mathrm{mvm}$ and


FIGURE 5 Traffic accident rates, 1985 through 1987.
$0.26 / \mathrm{mvm}$, respectively. The plot of hourly accident rates shown in Figure 6 suggests two findings. First, the major component of the peaking of accident rates in the late evening and early morning hours is because of single-vehicle accidents; this certainly makes sense, because the lower traffic volumes during these hours lessen the opportunity for multiple-vehicle conflicts. Second, the rates of single- and multiple-vehicle accidents during daylight hours are similar; with the exceptions of the hours beginning at 1:00 and 2:00 p.m., the singlevehicle accident rate is always greater than the multiplevehicle accident rate.

Numerous characteristics differ between daytime and nighttime operation, especially on rural highways. On the one hand, enforcement presence is much lower on these roads at night, thus reducing the likelihood that a reportable accident will be reported. On the other hand, there is ample evidence that the driving environment changes during the hours of darkness. Visibility is obviously worse, and drivers during the low-volume hours are more likely to be tired or impaired.

The role of single-vehicle accidents and the complexities of nighttime operation vis-à-vis hourly volumes in contributing to the substantial accident rate variations at low flow rates is beyond the scope of this project, which is more concerned with the relationship at high volumes. Despite the pattern shown in Figure 5, it may be that, over the volume range of


FIGURE 6 Single- and multiple-vehicle accident rates.
interest to the analyst, accident rates on rural New Mexico roads are independent of hourly volumes.

A flaw is inherent in the results shown in Figures 2-6. Specifically, the 44 study sites include a mix of two-lane and four-lane roadways, with obviously different volumes and capacities. This mixture of the two types of sites tends to mask the effects both at low and high volume levels. To resolve this shortcoming, the data base was divided into two sets: (a) a group of 24 low-volume sections with ADT values less than $4,500 \mathrm{vpd}$ and (b) a group of 20 high-volume sections with ADT values greater than 4,500 vpd. With two exceptions, this grouping divided the data base into two-lane and fourlane facilities. Graphs analogous to Figures 4 and 5 were prepared for each group.

The 24 low-volume sites had an average ADT of approximately 2,200 . The total number of accidents was 567 , for an average accident rate of $0.97 / \mathrm{mvm}$. The hour beginning at 6:00 p.m. had the highest number of accidents (55); on the basis of a mean volume of 130 vph , the accident rate during this hour was $1.61 / \mathrm{mvm}$. With the exception of this hour, accident rates for the hours from 6:00 a.m. to 7:00 p.m. were all below the daily average. As shown in Figure 7, peak accident rates occurred during the early morning hours. The pattern in this figure is similar to that shown in Figure 2, although the rates are substantially higher. In Figure 8, the accident rates are plotted as a function of average hourly volumes, which varied from 13 at 3:00 a.m. to 172 at 4:00 p.m. As a coarse approximation, the two-directional capacity of these sections could be estimated at $1,800 \mathrm{vph}$; the abscissa in Figure 8 therefore corresponds to a range of $v / c$ ratios from 0 to approximately 0.10 . The trend of decreasing accident rate with increasing average hourly volume is evident, with the only significant exception occurring at 6:00 p.m.

The 20 high-volume sites, with an aggregate length of 185 mi , had an average ADT of nearly 11,300 . On the basis of the total of 1,439 accidents during the period 1985 through 1987, the average accident rate was 0.63 . In contrast to the two-lane roads, the peak number of accidents (87) occurred at 7:00 a.m., when the accident rate was 0.75 . The hourly variation in accident rates shown in Figure 9 displays the familiar pattern of below-average rates during the daytime and above-average rates at night. However, peak accident rates are only half the values for the low-volume sections.


FIGURE 7 Accident rates on low-volume sections.


FIGURE 8 Accident rates on low-volume sections.


FIGURE 9 Accident rates on high-volume sections.

The relationship between accident rates and average hourly volume is shown in Figure 10. In this figure, the maximum peak hourly volumes of 821 vph (at both 4:00 and 5:00 p.m.) correspond to a $v / c$ ratio value in the range of 0.15 to 0.20 . Over the range of moderate traffic volumes shown in this figure, there is no indication that accident rates increase as the $v / c$ ratio increases.

On the basis of the conditions studied and the relationships displayed in Figures 2-10, it is possible to draw the following intermediate conclusions:

1. Accident rates tend to be below the daily average during the daylight hours and are substantially greater during the low-volume hours of darkness.
2. Over the range of average hourly volumes studied on rural New Mexico highways, accident rates decrease with increasing volumes. However, when the very low-volume hours are ignored, it may be that accident rates are independent of flow rates.
3. The major difference between daytime and nighttime accident rates appears to be attributable to the unusually high single-vehicle accident rates during hours of low traffic volume. In addition, the single-vehicle accident rate is, in most cases, higher than the multiple-vehicle accident rate.


FIGURE 10 Accident rates on high-volume sections.
4. Although New Mexico's permanent count stations are located at representative locations on rural two-lane roads, they do not embrace a full range of hourly volumes and $v / c$ ratios. The highest average hourly volume at any of the twolane sections is 365 vph , corresponding to a $v / c$ ratio of approximately 0.20 . Because of the limited latitude of $v / c$ ratios, it is not possible to use the New Mexico data base to develop a meaningful relationship for these facilities over the full range of $v / c$ ratios.
5. Accident rates on New Mexico freeways and other fourlane highways are about 65 percent of those on two-lane roadways. However, because of their significantly higher volumes and marginally higher $v / c$ ratios, the multilane highways provide a better opportunity for establishing the relationship between accident rates and $v / c$ ratios.

These findings are not particularly surprising in hindsight. However, a better understanding of this subject requires sections of road with higher $v / c$ ratios. The next task, therefore, involved a more detailed study of the highest volume fourlane sections in an effort to extend the relationship shown in Figure 10.

## HIGH-VOLUME FOUR-LANE ROADWAYS

The trade-offs between the long-term examination of highvolume sections and the shorter-term evaluation of a more extensive portion of the roadway system were described previously. Although overall preference was given to the examination of a larger number of sections, it was necessary to focus on the highest volume sites if information was to be developed regarding the upper ranges of the $v / c$ ratio. It was recognized at the onset that an individual $10-\mathrm{mi}$ section, even though examined for a 3-year period, would exhibit significantly more variation in volume and accident parameters than the larger group of high-volume sections shown in Figures 9 and 10 . Nevertheless, this course of action was the only one feasible within the constraints of the existing data base. As a result, the three multilane, permanent count stations with the highest ADT values were examined in greater detail. Two of these sections are on the Interstate highway system, whereas the third is on a federal-aid primary system. The characteristics of these sections are presented in Table 1.

TABLE 1 SUMMARY OF THREE HIGH-VOLUME SITES

| Station | 99 | 270 | 520 |
| :---: | :---: | :---: | :---: |
| Route | US 84 | I-40 | I-10 |
| Milepost | 168.61 | 176.07 | 155.25 |
| 1988 Volumes |  |  |  |
| ADT | 26214 | 19226 | 19185 |
| Ave Min Hr | 54 | 219 | 204 |
| Ave Max Hr | 2354 | 1389 | 1345 |
| \%HC (1987) | 4 | 25 | 22 |
| Highest Hourly Volumes |  |  |  |
| 50th Highest | 2817 | 1605 | 1613 |
| 30th Highest | 2869 | 1640 | 1642 |
| 10th Highest | 3157 | 1740 | 1740 |
| Highest | 5064 | 1978 | 1951 |
| Capacity (Approx.) |  |  |  |
| Design Speed | 60 | 70 | 70 |
| PHF | 0.9 | 0.9 | 0.9 |
| Driver | Commuter | Weekend | Weekend |
| Terrain | Rolling | Rolling | Level |
| Capacity/dir | 2864 | 1851 | 2808 |
| Capacity | 5700 | 3700 | 5600 |
| V/C Ratios |  |  |  |
| Ave Min Hr | 0.01 | 0.06 | 0.04 |
| Ave Max Hr | 0.41 | 0.38 | 0.24 |
| 50th Highest | 0.49 | 0.43 | 0.29 |
| 30th Highest | 0.50 | 0.44 | 0.29 |
| loth Highest | 0.55 | 0.47 | 0.31 |
| Highest | 0.89 | 0.53 | 0.35 |

As shown in Table 1, the ADT on these high-volume sections ranged from 19,200 to $26,200 \mathrm{vpd}$. On the two Interstate sections, more than 20 percent of the traffic consisted of heavy commercial (HC) traffic, whereas on the divided four-lane rural arterial, only 4 percent of the traffic was HC. This particular characteristic, along with the nature of the terrain, has a significant effect on highway capacity. In general, d:ivers on the Interstate sections are relatively unfamiliar with the roadway, whereas those near Station 99 tend to be commuters, which has an effect on highway capacity. Data are also shown for selected highest hourly volumes. The highest hourly volume is the two-way traffic during one of the 8,760 clock hours (e.g., 4:00 to 5:00 p.m.) that was the highest for the entire year. A similar definition applies to the 10th, 30th, and 50th highest hours. As indicated by the data, the section on US-84 has some rather extreme peaking characteristics, with the $\nu / c$ ratio at the highest hour being 0.89 ; by contrast, the comparable figures on the Interstate sections are 0.35 and 0.53 . Some level of stability is reached, however, when the 30th highest hourly volume, a value often considered in highway design, is examined. At this point, the $v / c$ ratios range from approximately 0.30 on I-10 to 0.50 on US-84. As shown in Table $1, v / c$ ratios change only slightly between the 30th and the 50th highest hours. The data reveal that even if a study is restricted to the highest hourly volumes at the highest ADT sites, $v / c$ ratios at New Mexico's rural permanent count stations rarely exceed 0.5 .
From this study's perspective, it is difficult to evaluate the extreme points on the $v / c$ curve. If, for example, the highest hour of the entire year at Station 99 (5:00 to $6: 00$ p.m. on
the last Monday in August) had no accidents during 1985 through 1987, the resulting accident rate would be $0.00 / \mathrm{mvm}$. On the other hand, with one accident during this particular hour, the accident rate would be $6.58 / \mathrm{mvm}$; intermediate rates are not possible. In fact, none of the 147 accidents of Station 99 during 1985 through 1987 occurred between 5:00 and 6:00 p.m. on the last Monday in August. Looking at a broader slice of time, all of the hours from noon to 7:00 p.m. on the last Monday in August were among the year's 30th-highestvolume hours. During the 3-year study period, there were no accidents during any of these hours on the last Monday in August. Continuing one step further, there was only one accident during these 7 hr on any Monday in August, yielding an accident rate of 0.45 versus an overall rate of 0.51 at this site. Finally, during all hours of the 13 August Mondays in 1985 through 1987, there were two accidents on this section, for a rate of 0.44 . The conclusion from this microanalysis of the highest volume section is that evaluation at this level can produce sporadic and inconsistent results.

As a compromise between the earlier analysis that grouped 20 four-lanc facilities and the preceding analysis that examined specific hours with the highest volume, an intermediate approach was taken. The three high-volume sites characterized in Table 1 were examined individually and collectively. Figures 11-13 show the relationship between accident rates and traffic volumes at the three sites. Although the best-fit lines in these figures are generally similar to those seen in the earlier graphs, the data exhibit a higher degree of scatter because, with smaller amounts of travel at a specific site, a difference of one or two accidents can cause a substantial


FIGURE 11 Accident rates on US-84.


FIGURE 12 Accident rates on I-10.


FIGURE 13 Accident rates on I-40.
change in the accident rate. These wide variations are moderated to some extent when the accident and travel data from the three sites are combined. As shown in Figure 14, data from the three sites allow the average accident rate to be evaluated for average hourly volumes as high as $1,700 \mathrm{vph}$. As a rough approximation, this average volume level corre-


FIGURE 14 Three high-volume count stations.
sponds to a $v / c$ ratio of 0.34 . The inevitable conclusion is that, over the range of volumes and $v / c$ ratios found at the highestvolume rural four-lane roadways, accident rates decrease sharply as average hourly volumes increase to about 900 vph $(v / c=0.18)$. Between hourly volumes of 900 and $1,700 \mathrm{vph}$, accident rates level off at approximately 70 percent of the overall average accident rate. It is not possible to determine from these data if a further increase in traffic volumes would be accompanied by an increase in accident rates.

## CONCLUSIONS AND RECOMMENDATIONS

An attempt was made to determine if a relationship exists between hourly accident rates and the ratio of traffic volume to capacity. Knowledge of any such relationship would help engineers and planners assess the safety implications both of projected traffic growth on existing highways and of highway improvements designed to increase capacity. The tool could therefore be a valuable supplement to current analysis techniques.
This objective was not achieved because peak traffic volumes on the rural highways examined are rarely as high as 50 percent of capacity. Although the variation in accident rates at low flow rates was documented, conclusions were not reached for the potentially more critical case of high $v / c$ ratios. For such highways, it is probably impossible to resolve this issue.
The highest accident rates were shown to occur at low volumes. This finding was expected because these conditions occur at night, when accident rates are known to be higher. But it also raises a related issue concerning the type of accident that should be included in the accident rate. It was assumed that vehicle conflicts, and in turn accidents, would increase as congestion increased beyond some threshold value. However, multiple-vehicle rather than single-vehicle accidents would increase. Figure 6 showed that the peaking of accident rates during the low-volume hours of darkness was caused primarily by the occurrence of single-vehicle accidents, although few crashes of this type can be attributed to congestion. Further study of this subject might achieve better results if restricted to multiple-vehicle crashes.

The New Mexico State Highway and Transportation Department has implemented a set of procedures that over time will improve the quality of traffic volume data collected at short-term count locations. The availability of a more extensive data base, possibly including sites with higher $v / c$ ratios, might provide an opportunity for further study of this topic. The other major variable considered in this study was traffic accident experience. The techniques used highlight the importance of high-quality accident data, especially with respect to crash location.
Significant progress was made on several major components of the problem. General trends were identified, and data problems that need to be addressed before further progress can be made were highlighted. Two preliminary studies, described in the project report (14), examined the feasibility of applying the techniques discussed herein to urban intersections and freeway sections. Although there are potential data problems with both of these highway elements, they both provide the opportunity to evaluate operation at high $v / c$ ratios. A proposed study will attempt to evaluate the relationship between accident rates and $v / c$ ratios at high-volume intersections in the Albuquerque area.

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# Relationship of $65-\mathrm{mph}$ Limit to Speeds and Fatal Accidents 

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#### Abstract

A time series analysis was performed on fatal accidents, injury accidents, vehicle miles traveled, and vehicle speeds over the 5 years preceding and 1 year following the increase in the national maximum speed limit (NMSL) allowed during the spring of 1987 on rural Interstate highways. In the states that raised their limits to 65 mph , speeding on rural Interstates increased by 48 percent and fatal accidents increased by 27 percent over projections based on previous trends. A 9 percent increase in speeding and a 1 percent increase in fatalities were observed on highways still posted at 55 mph . In the states that retained the $55-\mathrm{mph}$ limit, fatal accidents increased by slightly more than 10 percent both on rural Interstates and other posted highways coincident with the change in the NMSL. Speeding on the two classes of highways increased by 18 and 37 percent, respectively. The total increase in fatal accidents attributed to the raised speed limit, both in $65-\mathrm{mph}$ and $55-\mathrm{mph}$ states, was estimated at approximately 300/year. A shift of high-speed traffic to rural Interstates from other highways may have contributed to the changes occurring in the $65-\mathrm{mph}$ states. The increase in fatal accidents on $55-\mathrm{mph}$ non-Interstate highways in states that did not raise their limits may have been caused, in part, by the absence of such a shift.


The $55-\mathrm{mph}$ national maximum speed limit (NMSL) was imposed during the fuel crisis of 1974 as a conservation measure. It was credited with contributing to the decline in fatal accidents that followed its passage (1).

As fuel became readily available, many drivers began to question the need for a $55-\mathrm{mph}$ limit and the proportions of drivers who exceeded the limit began to creep upward. The greatest resistance to the lower speed limit came from the western states, where longer travel distances made an additional 10 mph a significant time saver. Some western states even threatened to raise their limits in defiance of the NMSL, even though this action would cause them to lose federal funds. Finally, in 1987 Congress voted to allow limits to be raised to 65 mph on rural Interstates, as well as on some other highways in specified experimental states. The law took effect on April 1 of that year and, by the end of the year, 38 states had raised the maximum limit.

NHTSA examined the effect of the raised limits on fatalities using time series analysis and reported that fatalities on rural Interstates were 14 percent higher than would have been expected on the basis of rates for those same highways over the previous 12 years (2). The report generated almost as much controversy as the change in speed limits itself. One criticism involved its use of annual accident totals, which grouped accidents during the 3 months preceding the law

[^10]change with those occurring during the remainder of the year. Critics contended that the upswing in accidents over those 3 months was evidence of an upward trend that was unrelated to the change in law and, therefore, should not be credited to the increased limit but rather subtracted from it. Another criticism was that the NHTSA analysis failed to account for the increase in fatalities that took place in states that retained the $55-\mathrm{mph}$ limit (although the time series model actually did so).

In 1988, the National Public Services Research Institute undertook a study to examine the impact of the $65-\mathrm{mph}$ limit on fatal accidents by (a) using monthly accident data to more precisely determine the time relationships involving changes in accidents and changes in the law, (b) studying changes in speeds as a variable intervening between the law change and accidents, (c) analyzing the extent to which the effects of the speed change on accidents varied with the states' characteristics, and (d) studying the effect of uniform versus dual speed limits on accidents to trucks and other vehicles (3).

The following discussion is limited to the relationship between the law change and fatal accidents. However, speed is examined as a variable mediating this relationship.

## METHOD

The increase in the NMSL was evaluated by comparing accident and speed data for $65-\mathrm{mph}$ versus $55-\mathrm{mph}$ states for the 5 years preceding the increase in the speed limit and the year following the increase. In order to separate the effects of the speed limit change from those of other changes that might have occurred at the same time, accident and speed data were also examined for a set of comparison highways, that is, highways on which the speed limit remained the same. The comparison highways included rural Interstates in those states that did not change the speed limit as well as other $55-\mathrm{mph}$ highways (urban Interstates and rural non-Interstates) in all states.

## Study Variables

The attempt to assess the effect of the $65-\mathrm{mph}$ limit on accidents involved speed laws, fatal accidents, and speeds.

## Speed Laws

The independent variables consisted of changes in state laws permitting increased speeds on rural Interstates. The variable
involved two categories: (a) those states that increased the speed limit to 65 mph , referred to as $65-\mathrm{mph}$ states, and (b) those that maintained the $55-\mathrm{mph}$ speed limits, referred to as $55-\mathrm{mph}$ states. For this study, the $65-\mathrm{mph}$ states were confined to 38 states that changed their speed limits during the period between April and June 1987. The six states that changed their speed limits at a later date during the year were excluded. Also excluded were four jurisdictions that lacked significant amounts of rural Interstate mileage. One of the states included in the $55-\mathrm{mph}$ group (Virginia) ultimately increased its speed limit, but only after the 1-year followup study period was over.

Among the 38 states in the $65-\mathrm{mph}$ sample, 20 changed speeds only on rural Interstates. These so-called "pure" states are emphasized here. Six states raised the limit only for automobiles, creating dual limits, and another six states raised limits for highways other than rural Interstates on an experimental basis. Finally, California did not raise limits on all eligible highways.

## Fatal Accidents

Fatal accidents were used as the primary dependent variable because of their ready availability and suitability for analysis. Although they represent only about 0.2 percent of all accidents, they are routinely reported to and recorded by the NHTSA Fatal Accident Reporting System (FARS). Injury and property damage accidents, although more numerous, are available only from individual states, are often maintained in a form unsuitable to research, vary in reporting thresholds from one state to another and one year to another, and often are not recorded as occurring on 65 - or $55-\mathrm{mph}$ highways. Fatal accidents were chosen over fatalities as being a more stable statistic; fatalities are influenced by the number of individuals in a vehicle, a factor that is not directly related to speed limits.

## Speeds

The use of speed data was important in distinguishing the extent to which changes in fatal accidents are accompanied by, and therefore are possibly attributable to, changes in speed. Speed data are routinely obtained on a sample of $55-\mathrm{mph}$ highways in all states under the speed monitoring program, upon which eligibility for highway trust fund monies is ascertained. Although it was no Ionger necessary to monitor speeds on rural Interstates on which speeds had been raised to 65 mph , many states continued to do so.

The speed data furnished by the states to FHWA are available only in summary form and were insufficiently detailed for the needs of the study. It was therefore necessary to obtain the speed data from the states themselves. Collection of data was obviously limited to those states that had continued to monitor speeds on rural Interstates, had the data disaggregated by type of roadway, had maintained a file of speed data from previous years, had the data in a form capable of being readily accessed, and were willing to make the data available. These conditions prevailed in 16 states, of which 9 had raised their speed limits and 7 had maintained the $55-\mathrm{mph}$ limit.

Because speed data are accumulated and analyzed once each quarter, data were only available on a quarterly basis. The specific speed variable selected for analysis was the percentage of drivers who exceeded 65 mph . This value was preferred over mean speed because the high speeds are the primary contributors to fatal accidents. The high proportion of drivers already exceeding the $55-\mathrm{mph}$ limit created a ceiling effect that made 65 mph the better limit for analytic purposes.

## Data Analysis

Speeds form a basis of a state's eligibility for federal funds; thus, the accuracy of speed data can certainly be questioned. However, any bias that exists should be fairly constant over time as far as $55-\mathrm{mph}$ highways are concerned. In addition, there is no reason to expect changes in biasing factors for 65 mph highways, because the states that continued to collect speed data on such highways did so at the same location using the same procedures.

The primary method of analyzing data was time series intervention analysis, which involves analyzing a series of quantities over time to determine if the pattern conforms to that expected as a result of an intervention, such as a change in the speed limit. In analyzing the effect of speed limit changes, the quantities of primary concern were accidents and drivers who exceeded 65 mph . Accidents were studied on a monthly basis and speeding on a quarterly basis.

As explained, the time series for accident and speed data involved the period 5 years preceding the law change and 1 year following it. The five previous years provided an adequate basis for establishing long-term trends. Going back any further was contraindicated by the major shifts in accident and speed trends that took place before 1982. The inclusion of earlier years would have made it difficult to find a suitable model for the preintervention time series.

The specific method of analysis used was the Box-Tiao ARIMA intervention analysis. This method provides the ability to estimate linear models of change over time despite the presence of noise caused by seasonal or other extraneous forms of variation. In the particular form of time series used, a model of the time series representing speed or accident data was compared with a time series for a dummy variable representing the law change (a variable with 0 values leading up to the point of intervention and 1 values afterward). Through regression analysis, it was possible to measure the degree to which variation in the accident or speed time series could be accounted for by the law change variable.

## RESULTS

## Fatal Accidents

The time series of fatal accidents is shown in Figure 1. The left column provides the series for data gathered from the pure $65-\mathrm{mph}$ states, and the right column represents all of the $55-\mathrm{mph}$ states. In each column, the top series represents rural Interstates, the second series represents other $55-\mathrm{mph}$ highways, and the third series represents the ratio of rural Inter-
"Pure" 65 MPH States Rural Interstate Fatal Accidents

"Pure" 65 MPH States
Other Highway Fatal Accidents

"Pure" 65 MPH States Ratio of Rural Intst.s to Other Highways


55 MPH States
Rural Interstate Fatal Accidents


55 MPH States Other Highway Fatal Accidents


55 MPH States
Ratio of Rural Intst.s to Other Highways


FIGURE 1 Monthly time series of fatal accidents by category of state and type of highway.
states to other $55-\mathrm{mph}$ highways. All of the accident time series are seasonally adjusted.

In $65-\mathrm{mph}$ States
The time series that should reflect the effect of the increased speed limits is that shown in the upper left corner-rural

Interstates in $65-\mathrm{mph}$ states. There is an apparent increase in fatalities in the early part of 1987 and a leveling off during the middle of the year. However, the increase appears to begin in mid-1986 rather than coinciding with the change in the NMSL, which occurred in the spring of 1987. The question is whether the leading edge of the upswing represents a true increase in fatalities or merely a return from an earlier
downswing. The time series intervention model shows a significant increase in fatal accidents ( $t=5.82, p<0.01$ ), estimated at 14.6 per month, for a yearly total of 176 fatal accidents associated with the increased posted speed limit. This result amounts to a 27.1 percent ( $\pm 9.4$ percent) increase over those that would have occurred if there had been no change in speed limit. (All confidence intervals correspond to the 95 percent level.)

Another uncertainty is whether the increase in fatal accidents on $65-\mathrm{mph}$ posted highways was caused by the speed limit change or was the result of other variables. One series that might be expected to reveal the effects of other variables would be that representing fatal accidents that occur on 55mph highways in $65-\mathrm{mph}$ states - the second series in the left column of Figure 1. This series does not indicate any effect of the change in the NMSL. A model of the time series shows an increase of only 0.6 percent ( $\pm 4.5$ percent), a difference that is not statistically significant $(t=0.28, p>0.10)$. If the increase in fatalities on $65-\mathrm{mph}$ highways was not caused by the higher speed limit, then it was caused by any factor that would also have affected fatal accidents on $55-\mathrm{mph}$ highways in the same states.
That the increase in fatal accidents on rural Interstates existed apart from any change on the $55-\mathrm{mph}$ highways is evident in the time series for the ratio of fatal accidents on the two highways that appears in the lower left position in Figure 1. The increase in the ratio is similar to that observed on the rural Interstates, amounting to a 26 percent increase ( $t=$ 4.01, $p<0.01$ ).

## In 55-mph States

It is possible that the increase in fatal accidents on rural Interstates was caused by some set of variables (other than the speed limit change) that affects fatal accidents only on rural Interstates. Such an effect might appear in the time series of fatal accidents on rural Interstates in those states that maintained the $55-\mathrm{mph}$ limit. This series is shown in the top right chart in Figure 1. There appears to be a small increase in the spring of 1987, and a leveling off thereafter. A model of the time series shows a statistically significant $(t=4.08, p<0.01)$ increase of 20 fatal accidents per year, or a 10.4 percent $( \pm 5.1$ percent) increase. A similar increase in fatal accidents early in 1987 also appears in the time series for fatal accidents on other highways in the $55-\mathrm{mph}$ states. A time series model estimates the increase at 12.7 percent ( $\pm 6.5$ percent), which is also statistically significant ( $t=4.03, p<0.01$ ). Because the raw number of accidents on other $55-\mathrm{mph}$ highways is much greater than the rural Interstates toll, a similar percentage increase amounts to a much larger number of accidents-an increase of 295 fatal accidents per year.

The variable that produced the increased speeds in the 55mph states seems to have affected rural Interstates and other $55-\mathrm{mph}$ highways equally. The similarity in effects is evident in the ratio of fatal accidents on rural Interstates to those on other highways (bottom right), which seems to represent random variation. No significant change was associated with the change in the NMSL $(t=0.52, p>0.10)$.

## In States with Mixed Limits

The $65-\mathrm{mph}$ category in Figure 1 included the pure states only. It did not include the six states with dual limits for trucks and passenger vehicles, the six states that were allowed to increase speed limits experimentally on some segments of rural nonInterstate highways, or California, which maintained the 55mph limit on 23 percent of eligible Interstate highways. Each of these categories yielded time series that were similar to those observed in the pure $65-\mathrm{mph}$ states, showing substantial and significant increases in fatal accidents on rural Interstates but insignificant changes on non-Interstate highways.
When all of the $65-\mathrm{mph}$ states were combined, a statistically significant ( $t=6.671, p<0.01$ ) increase of 21.8 percent, or 313 , fatal accidents per year occurred on rural Interstates, whereas an insignificant change ( $t=0.76, p>0.10$ ) occurred on the $55-\mathrm{mph}$ posted highways. A 21 percent increase in the ratio of rural Interstates to $55-\mathrm{mph}$ posted highways is also statistically significant ( $t=6.37, p<0.01$ ).

## Vehicle Miles of Travel (VMT)

An increase in a variable other than speed that might have contributed to the increase in fatal accidents on $65-\mathrm{mph}$ posted highways could be the increase in VMT that was not shared by $55-\mathrm{mph}$ highways in the same states. Such an increase would result if the increased speed limit caused drivers to shift to the rural Interstates from other highways.

In $65-\mathrm{mph}$ states, annual VMT on rural Interstates increased by 8.1 percent between 1986 and 1987, compared with a 6.7 percent increase on non-Interstates-a difference of 1.4 percent. Although this increase is small, it could have had a major impact on the number of fatal accidents if it consisted primarily of drivers operating at high speeds. In this case, however, any effect would be attributed to the higher speeds rather than simply to increased mileage.

## Summary

Fatal accidents appear to have increased sharply on rural Interstates in the $65-\mathrm{mph}$ states coincident with the increased speed limit. No significant increase in fatal accidents occurred on highways posted at 55 mph in those states. This result was expected. What was not expected, and cannot readily be explained by the change in the NMSL, is the increase in fatal accidents on $55-\mathrm{mph}$ posted highways in the states that did not raise their speed limits. Although this increase was less than half the magnitude of the increase in states that raised the limit, these increases in fatal accidents are statistically significant.

## Speed Data

The time series for the percentage of drivers who exceeded 65 mph are shown in Figure 2. These series parallel those for fatal accidents shown in Figure 1 but are based on quarterly rather than monthly data.


FIGURE 2 Quarterly time series of percentage of drivers who exceeded 65 mph by category of state and type of highway.

## In 65-mph States

The nine $65-\mathrm{mph}$ states from which speed data were obtained included two pure $65-\mathrm{mph}$ states, two experimental states, four states with dual limits, and California. However, an analysis not presented here indicated that all four categories of $65-\mathrm{mph}$ states evidenced the same pattern of change in fatal accidents. Indeed, within the sample of nine states, fatal acci-
dents increased by 20.6 percent on rural Interstates ( $t=4.33$, $p<0.001$ ) and by 0.2 percent on non-Interstates ( $t=0.14$, $p=0.789$ ). The speed changes observed in the nine-state sample may therefore be generalized to the remaining $65-\mathrm{mph}$ states.
A marked increase in the proportion of drivers who exceeded the $65-\mathrm{mph}$ limit on rural Interstate highways is readily observable in the top left chart in Figure 2. Expressed in relation
to the baseline series, the increase is equal to 48.2 percent ( $\pm 18.4$ percent) and is statistically significant ( $t=4.97$, $p<0.01$ ). A multivariate time series showed the increase in speeding to be significantly related to the increase in fatalities ( $t=3.97, p<0.001$ ).

The second chart in the left column of Figure 2 displays the speed series for $55-\mathrm{mph}$ highways in the same states. Clearly, speeds on other $55-\mathrm{mph}$ highways did not increase as sharply as did those on the rural Interstates. Whether there is an increase at all is debatable. If the series is viewed as beginning to level off or decline to an earlier level, then the slight increase through 1987 might be considered an increase in the amount of speeding. On the other hand, 1987 might well be viewed as part of a leveling off and thus a decline relative to the upward trend of earlier years. The time series model estimates the change at a 9.1 percent ( $\pm 6.7$ percent) increase, which is statistically significant ( $t=2.67, p<0.01$ ).

In any case, the increase in speeders on roads posted at 65 mph far exceeded that experienced on roads that maintained the $55-\mathrm{mph}$ limit. This effect is shown in the ratio of the two sets of highways, which indicates an increase of 28 percenta result that is highly significant $(t=3.27, p<0.01)$. The three times series for speeding in $65-\mathrm{mph}$ states parallel those for fatal accidents, lending support to the idea that speed was involved in the increased number of fatal accidents on rural Interstates. The time series do not show drivers' increasing their speed from 55 to 65 mph ; rather, they show increased numbers of drivers exceeding 65 mph .

## In 55-mph States

The right side of Figure 2 displays the time series for all of the states that maintained the $55-\mathrm{mph}$ limit. The results are not dissimilar to those for fatal accidents-indicating slight increases on both classes of highways. The increase in speeders on rural Interstates is estimated by the model as 18 percent ( $\pm 17.7$ percent) $(t=2.02, p<0.05)$ and that on other 55 mph highways as 37 percent $( \pm 19.9$ percent $)(t=3.66$, $p<0.01$ ). The ratios of speeds on the two categories of highways have no meaningful pattern.

Again, the results suggest that the apparent increase in fatal accidents was associated with increased numbers of speeding drivers.

## SUMMARY AND CONCLUSIONS

Increasing the speed limit to 65 mph on rural Interstates was associated with a marked ( 27 percent) and statistically significant increase in the number of speeders and fatal accidents on the highways affected by the change. On highways that retained a $55-\mathrm{mph}$ limit, there was no increase in fatalities and a relatively small ( 9 percent) increase in speeders. There seems little doubt that the increase in speeding that occurred in the spring of 1987 led to an increase in fatal accidents.

What caused the increase in speeding? If it was the increase in the NMSL, why did speeding go up both on rural Interstates and on other highways in states that maintained the $55-\mathrm{mph}$ limit? The increases were less than half of those observed on $65-\mathrm{mph}$ highways but were statistically significant.

The increase in speeds and accidents in the $55-\mathrm{mph}$ states might be attributed to factors other than the change in speed limit, and those factors could be adjusted for by subtracting the change found in the $55-\mathrm{mph}$ states from that observed in the $65-\mathrm{mph}$ states. Doing so would reduce the estimated increase in fatal accidents attributed to the speed limit by approximately one-half, placing it close to the estimate of 14 percent provided by NHTSA. However, if the increases in speeds and fatalities on highways in the $55-\mathrm{mph}$ states were caused by a factor other than the change in law, why did they coincide with the law change and why did they not also appear on $55-\mathrm{mph}$ posted roads in the $65-\mathrm{mph}$ states?

One explanation of the findings could be a change in the public's attitude toward the $55-\mathrm{mph}$ limit that coincided with the change in law. Whether the law change led to the shift in attitudes or vice versa, greater numbers of drivers in $55-\mathrm{mph}$ states began to operate at higher speeds at the time Congress voted to ease the $55-\mathrm{mph}$ limit.
If changes in attitude toward the $55-\mathrm{mph}$ limit did cause the increase in speeds and fatal accidents on $55-\mathrm{mph}$ highways in states that did not raise the limit, why was there no significant increase in fatal accidents on $55-\mathrm{mph}$ highways in those states that did raise the limit? One possibility is that drivers who wanted to drive at 65 mph in the states that raised their limits could legally do so by using rural Interstates. Although the relative change in vehicle mileage to rural Interstates in $65-\mathrm{mph}$ states was quite small ( 1.4 percent), it could have had a significant impact on speeds and fatal accidents if it consisted primarily of high-speed traffic. The 27 percent increase in fatal accidents on rural Interstates might then have kept an even larger increase from occurring on the more numerous and more heavily traveled $55-\mathrm{mph}$ segments.

The idea that raising speed limits on rural Interstates drew high-speed traffic from other roads and produced a net benefit is highly speculative. However, it seems reasonable to assume that any attempt to hold speed limits on rural Interstates at 55 mph at a time when large segments of the driving public believe they are safe at 65 mph will only be successful when enforcement is sufficient to maintain a high degree of compliance with speed limits. In the absence of such enforcement, it may be better to raise the speed limits on those highways most able to accommodate higher speeds than to allow drivers to speed on all highways.
It is clear that an increase in speeding concurrent with the change in the NMSL led to an increase in fatalities. However, it is unclear whether the speeding resulted from the law change or whether both resulted from fundamental changes in the public's definition of an acceptable speed. In any case, merely maintaining a $55-\mathrm{mph}$ limit did not suppress speeding or fatal accidents. Indeed, with respect to fatal accidents, attempting to maintain a $55-\mathrm{mph}$ limit on all highways may have been counterproductive.

From the results of the study, the following conclusions are offered:

1. Raising speed limits to 65 mph coincided with an estimated 48 percent increase in the number of speeders on rural Interstates, resulting in a 22 percent increase in fatal accidents (approximately 300 fatal accidents per year).
2. In the $65-\mathrm{mph}$ states, neither the number of speeders
nor the number of fatal accidents on $55-\mathrm{mph}$ bighways increased following the increase in speed limits.
3. In states that retained the $55-\mathrm{mph}$ limit, fatal accidents on rural Interstates and other $55-\mathrm{mph}$ highways increased by an estimated 10 and 13 percent, respectively. This increase also amounts to an estimated increase of approximately 300 fatal accidents per year.
4. Although the increased number of fatal accidents in 55mph states cannot be attributed directly to the change in speed limit, it appears to be the result of significant increases in the numbers of speeders coinciding with the change in speed limit.
5. In the face of widespread noncompliance with the 55mph limit, raising the limit on rural Interstates may benefit safety by diverting some speeders to the highways best able to accommodate them.

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# Preliminary Assessment of the Increased Speed Limit on Rural Interstate Highways in Illinois 

Charanjit S. Sidhu


#### Abstract

In May 1987, a $65-\mathrm{mph}$ speed limit was posted on rural Interstate highways in Illinois. The effect of the change in vehicle speeds on the incidence of accidents is assessed for the first year. The method consisted of using 5 years of data collected before the new speed limit and subjecting these data to linear regression to project the number of accidents had there been no change in the posted speed limit. The projected number of accidents of each type was then compared with the reported numbers for the period of the assessment. Average speeds of passenger vehicles on the rural Interstates increased from 59.8 mph during the preceding year to 61.8 mph during the initial 12 months of the increased speed limit. The number of fatal accidents (expected versus reported) increased on each of the three different types of highways ( 15.2 percent on rural Interstates, 20.3 percent on urban Interstates, and 2.9 percent on the primary system of highways). Most of the increase in fatal accidents on the rural Interstates may be attributed to the increase in fatal pedestrian accidents and fatal accidents involving drinking and driving. Results indicate that the higher posted speed limit in Illinois did not have a clearly noticeable or an obviously adverse effect on fatal accidents during its first year.


For over 13 years (since March 1974), the maximum posted speed limit in all states, including Illinois, was 55 mph . This limit was enacted by Congress to conserve fuel in response to the oil embargo of 1973. As a result, traffic slowed on all major highways and the total amount of travel declined in 1976 for the first time since 1946. These changes in speed and travel were accompanied by a decreasing number of traffic fatalities. As the fuel shortage disappeared, safety became the dominant issue surrounding the $55-\mathrm{mph}$ speed limit.

As compliance with this speed limit declined over the years and as fuel supplies became plentiful, the safety benefits of the speed limit also decreased. The early and middle 1980s were a period of intense debate, culminating with an assessment of the $55-\mathrm{mph}$ speed limit by TRB (1) at the direction of Congress. The findings and recommendations of this study apparently opened the door for a higher speed limit on the nation's rural Interstate highways. Toward the end of the 55mph limit (1985 through 1986), motorists' support for the 55mph speed limit was exceeded only by their disregard for it on the road.

On April 2, 1987, Congress, with the passage of the Surface Transportation and Uniform Relocation Assistance Act

[^11](STURAA), removed federal sanctions and permitted states to increase the posted speed limit to 65 mph on their rural Interstates. For all practical purposes, the $65-\mathrm{mph}$ posted speed limit became effective on Illinois rural Interstates on May 1, 1987, and it applied to passenger vehicles, buses, motorcycles, and trucks under 4 tons. The speed limit on these highways for trucks over 4 tons, motor homes, campers, and trailers was maintained at 55 mph .

## EVALUATION APPROACH

Since the second half of 1987, a number of reports assessing the effect of the $65-\mathrm{mph}$ speed limit on accidents on rural Interstates have been released by various agencies. Most of these reports are based on before and after comparisons of accidents on the highways affected by the new speed limit. The corresponding periods for which accident data are compared are usually no longer than 1 year. This approach is unsatisfactory because this method ignores the trend of accidents on the highways in question over the previous several years, before the transition to the higher specd limit. Further, most of these studies are limited to analysis of fatal accidents and ignore the effect of the higher speed limit on injury and property damage accidents.

The evaluation approach applied is based on 5 years of accident data for approximately $1,200 \mathrm{mi}$ of rural Interstates for the period when they were posted at 55 mph . With the application of regression analysis, the numbers of different types of accidents (fatal, personal injury, and property damage) are projected that are expected to occur for the period being assessed (May 1987 through April 1988) when the 65mph speed limit applied. The difference in accidents between the projected numbers and the actual or reported number is attributable to the change in the posted speed limit of such highways. Accident data for other Interstates and other roads (non-Interstate) with the $55-\mathrm{mph}$ posted speed limit have also been subjected to the same approach and analyzed. The source of these data is the computerized accident data files maintained by the Illinois Department of Transportation (IDOT), Division of Traffic Safety. Speed data for different highways were obtained from the IDOT Division of Highways, which operates automated speed-monitoring equipment on all types of roads throughout the state.

## CHANGE IN VEHICLE SPEEDS

Average speeds of passenger vehicles on rural Interstates varied from a low of 57.0 mph to a high of 60.7 mph (a range of 3.7 mph ) during the 5 -year period before the $65-\mathrm{mph}$ speed limit was posted in May 1987. The average speed for the second quarter of 1987 was 59.7 mph . Since introduction of the higher speed limit, the average speeds increased during the third and fourth quarters of 1987 to 62.0 and 62.3 mph , respectively. During the first quarter of 1988 , speeds reached 63.2 mph , declining in the second quarter to 62.1 mph .

The average speed for passenger vehicles on the urban Interstates declined somewhat for the period of the assessment, from 57.1 mph (April through June 1987) to 55.7 mph (April through June 1988). The corresponding 85th-percentile speeds for these vehicles also declined from 65.4 to 64.3 mph . Similarly, the proportion of these vehicles exceeding 70 mph decreased from 4.0 to 3.0 percent.

Over a period of 5 years, from the second quarter of 1982 through the first quarter of 1987, the average speed of passenger vehicles on the two-lane primary system (state highways) was approximately 54 mph . During the first year of the increased speed limit on rural Interstates, average speeds on state highways ranged between 56.4 and 56.8 mph , an increase roughly close to that experienced on rural Interstates.

More specifically, before-after changes in average speeds of passenger vehicles on different highway systems are presented in Table 1, and speed data are graphically shown in Figures 1-3.

## CHANGE IN ACCIDENTS

Generally speaking, accompanied by a 2 - to 3 -mph increase in average speeds of motor vehicles, accidents on the rural Interstates showed increases in each category. Similarly, fatal and personal injury accidents increased on urban Interstates; however, average speeds of vehicles on these highways had apparently declined by 1 to 2 mph . On the other hand, property damage accidents on urban segments declined, as expected, and the change was generally consistent with the change in average speeds. Except for fatal accidents on primary roadways, which increased slightly as expected, personal injury and property damage accidents apparently decreased by a slim margin, despite a small increase ( 1 to 2 mph ) in the average speeds of vehicles. The changes in accidents are presented in Table 2.

## INTERPRETATION OF RESULTS

A detailed examination of individual fatal accident reports from May 1987 through April 1988-for accidents on rural Interstate highways-reveals that the number of fatal pedestrian accidents and fatal accidents involving drinking and driving were 11 and 13 , respectively. Individual fatal accident reports were also examined for the $24-$ month period (May 1985 through April 1987) before the increase in posted speed limit. The corresponding average frequencies of such accidents in this earlier period were 7 (pedestrian accidents) and 9 (accidents attributed to drinking and driving), respectively. From these data, hardly any reason exists to believe that a change in the speed limit, one way or another, caused any of these fatal accidents. In all likelihood, the apparent increase in such accidents had little to do with the speeds of vehicles traveling on rural Interstates. If a portion of the before-versusafter increases in these two types of fatal accidents (total of $4+4=8$ instances) was unconnected with the posted speed limit, the gap between expected and reported fatal accidents would only become smaller. Therefore, any increase in fatal accidents on rural Interstate highways caused by the increased speed limit must indeed be smaller than that shown in the results.
The data for the severity of accidents on rural Interstate highways are also worth examining. Personal injury rates (per 100 million vehicle miles of travel) for the periods of interest are presented in Table 3.

A glance at these data shows that the personal injury rate during May 1987 through April 1988 is quite consistent with such rates in the previous years. If anything, the rate for the period of assessment is somewhat lower than that of the preceding year (May 1986 through April 1987). In addition, when the proportions of the most severe injuries (Class A injuries) are examined, the earlier conclusion regarding the severity of accidents remains unchanged, as presented in Table 4.
Further, when fatal accidents and personal injury accidents as proportions of total accidents were tested for differences (Chi-squared test, 95 percent confidence) for the before (May 1982 through April 1987) and after periods (May 1987 through April 1988), no significant differences were found between such proportions. In short, the severity of accidents on rural Interstates almost certainly did not worsen and stayed within limits of random variation.
In summary, the analysis of all relevant data and the interpretation of results support the view that the $65-\mathrm{mph}$ posted speed limit on rural Interstate highways in Illinois had appar-

TABLE 1 BEFORE-AFTER SPEED CHANGES FOR PASSENGER VEHICLES

|  | Average Speed (mph) |  |  |
| :--- | :--- | :--- | :--- |
|  | Before |  | After |
|  |  |  | May 1986-April 1987 |
|  |  | May 1987-April 1988 |  |
| Hural Interstates | 58.7 | 59.8 | 61.8 |
| Urban Interstates | 55.4 | 56.6 | 55.9 |
| Primary system | 54.0 | 55.1 | 56.1 |



FIGURE 1 Average speeds of passenger vehicles on rural Interstate routes by quarter.


FIGURE 3 Average speeds of passenger vehicles on primary routes by quarter.


FIGURE 2 Average speeds of passenger vehicles on urban Interstate routes by quarter.

TABLE 2 EXPECTED VERSUS REPORTED ACCIDENTS

| Accident Type | No. Expected | No. Reported | \% Change | Statistically Significant |
| :--- | ---: | :---: | :---: | :---: | :---: |
| (Rural Interstate Highways) |  |  |  |  |
| F.A | 46 | 53 | +15.2 | No |
| P.I. | 1,120 | 1,181 | +5.4 | No |
| P.D. | 2,483 | 2,647 | +6.6 | Yes |
| (Urban Interstate Highways) |  |  |  |  |
| F.A. | 64 | 77 | +20.3 | No |
| P.I. | 6,146 | 6,231 | +1.3 | No |
| P.D. | 15,239 | 14,105 | -7.4 | Yes |
| (Primary or State Roadways) |  |  |  |  |
| F.A. | 669 | 689 | +2.9 | No |
| P.I. | 50,626 | 49,945 | -1.3 | No |
| P.D. | 122,325 | 121,379 | -0.7 | Yes |

TABLE 3 PERSONAL INJURY RATES

| Tme Periods | Personal Injury Rates |
| :---: | :---: |
| May 1982 - April 1983 | 29.0 |
| May 1983 - April 1984 | 43.0 |
| May 1984 - April 1985 | 32.0 |
| May 1985 - April 1986 | 32.3 |
| May 1986 - April 1987 | 34.1 |
| May 1987 - April 1988 | 33.8 |

TABLE 4 PROPORTION OF SERIOUS INJURIES

| Time Periods | No. of "A" <br> Injurles | No. of Total <br> Injuries | \% of Total |
| :--- | :---: | :---: | :---: |
| May 1982 - April 1983 | 510 | 1313 | 38.8 |
| May 1983 - April 1984 | 745 | 1973 | 37.7 |
| May 1984 - April 1985 | 608 | 1543 | 39.4 |
| May 1985 - April 1986 | 600 | 1636 | 36.6 |
| May 1986 - April 1987 | 530 | 1816 | 29.1 |
| May 1987 - April 1988 | 572 | 1901 | 30.0 |

ently a small effect on fatal accidents during the first year. Similarly, even though personal injury accidents on these highways apparently increased, the observed change is within the limits of random fluctuation. On the other hand, property damage accidents on these highways increased beyond their expected level and the increase in such accidents is statistically significant.

The first year of the increased speed limit in Illinois does not appear to have had a clearly noticeable or an obviously adverse effect on the safety of rural Interstate motorists.

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## PART 2

## Pedestrian Studies

# Measurement of Pedestrian Flow Data Using Image Analysis Techniques 

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#### Abstract

Image analysis techniques are applied to measure number of pedestrians and their walking directions. A new algorithm, which consists of eight steps, is developed. An image device system is used to record pedestrian images in a hallway passage. An image subtraction procedure, thinning procedure, filling procedure, and Boolean-type operation are derived for the algorithm to process and analyze the images. Results show that image analysis has significant potential in the area of automatic measurement of pedestrian flow data. However, in this preliminary stage, the process has only limited success. For low- to average-density pedestrian traffic situations, the accuracy in measuring the number of pedestrians and their direction of travel is about 93 and 92 percent, respectively. The time complexity of the algorithm and the possibility of real-time analysis are also discussed.


The increasing use of pedestrian facilities such as building complexes, shopping malls, and airports in densely populated cities demands pedestrian flow data for planning, design, operation, and monitoring of these facilities. Pedestrian flow data are also needed to measure the demand for service, to locate areas in which new facilities are needed, and to justify and time pedestrian signals (1).

Pedestrian flow data consist of characteristics such as volume, density, speed, and direction. Pedestrian volume is the number of pedestrians that pass a perpendicular line of sight across the width of a walkway during a specified period of time. Density is the concentration of pedestrians within a walkway. Speed is the average walking speed, and direction is the walking direction of a pedestrian. Elements of density and direction are examined in the hope that significant results will lead the way to similar studies of the elements of volume and speed.

Currently, measurement of pedestrian flow data is often performed manually. For instance, manual determination of pedestrian volume requires one or more observers equipped with mechanical counters to record the number of pedestrians walking across an observation area (2). Manual counting is expensive and not suited to counting a large volume of pedestrians. Pedestrian data can also be obtained by videotaping traffic situations and then analyzing these permanent records in the laboratory (3). This method is still time-consuming, and positioning of the camera can be troublesome. Another way of measuring pedestrian flow is the automatic counter, which

[^12]consists of detector pads laid on the sidewalk and connected to a counting device (4). This device is probably the best volume determination system currently available, but this system is incapable of measuring other pedestrian flow data such as speed and walking direction. In addition, aerial photography has been used for gathering traffic data over large areas. Photographs of the study area are taken from an airplane and later analyzed using special eyepieces (5). However, aerial photography is an onerous and extremely time-consuming endeavor. Hence, a review of the literature indicates that a device is currently unavailable that can automatically collect and analyze pedestrian flow data.

A new system for collecting all types of pedestrian flow data will not be proposed. However, through the use of image analysis techniques, an investigation will be made of the feasibility of automatically measuring the number of pedestrians in an observation area and their walking direction. Application of image analysis techniques to collecting pedestrian flow data is relatively new. Hwang and Takaba (6) placed a number of detection points on the surface of a path. Using image analysis techniques, they counted the number of pedestrians walking in a common direction under the restriction that some separation exist between the pedestrians. However, Hwang and Takaba (6) did not study walking direction. Image analysis techniques are used and an algorithm is developed. Accuracy, complexity, and real-time analysis of this algorithm are also examined.

Dense multidirectional flow measurement encounters major problems when image analysis techniques are used. These problems are aggravated by the constant movement of legs, arms, and torsos and by the overlapping problems caused by the viewing angle. The human eye may even encounter difficulties when measuring that type of flow. Thus, only lowto average-density pedestrian flow situations are considered. Low-density pedestrian flow is equivalent to the flow situation under level of service (LOS) A or B specified in the 1985 Highway Capacity Manual (HCM) (7). Average-density flow situation is equivalent to the flow situation under LOS C or D in the 1985 HCM . In the 1985 HCM , average pedestrian space is greater than $130,40,24$, and $15 \mathrm{ft}^{2}$ per pedestrian for $\operatorname{LOS} \mathrm{A}, \mathrm{B}, \mathrm{C}$, and D , respectively.

## IMAGE ANALYSIS

## Image Analysis Techniques

Image analysis is a subject related to computer vision. An image is a two-dimensional array of pixels, obtained with a
sensing device that records the value of an image feature at all points. A pixel is a contraction of picture element, a dot or dash of light produced by an electron beam striking a phosphorescent surface of the cathode-ray tube (8). Images are converted into digital form for computer processing. For a halftone black-and-white image, every pixel can be assigned a grey value depending on its brightness. Grey values range from zero, indicating the dimmest level in an image, to 255, indicating the brightest level. The goal of image analysis is the construction of scene descriptions on the basis of information extracted from the digitized images or image sequences (9).

Over the past two decades, many techniques for analyzing images have been developed. The main applications of image analysis include document processing, microscopy, industrial automation, remote sensing, and reconnaissance. Since the mid-1970s, the U.S. Department of Transportation has been funding research on image processing applied to freeway surveillance at the Jet Propulsion Laboratory (JPL) in Pasadena, California. A wide-area detection system (WADS) (10) was developed for tracking vehicles within the area.
Image analysis techniques generally include four stages: image acquisition, data processing, feature extraction, and object recognition. Image acquisition consists of obtaining pedestrian images using a sensing device. Data processing removes all irrelevant information, such as the scene background, from the image. Next, important features such as the shape and size of objects can be extracted from the image. Finally, the number and walking direction of pedestrians can be obtained in the object recognition stage. Using this fourstage procedure, a new algorithm for measuring the number of pedestrians and their direction was developed.

## Image Device System

Figure 1 shows the structure of the five items used in the image device system. 'lhese five items are

1. Videocamera: A Sony Video-8 camera that uses 8 -mm videotapes was used.
2. Interface board: An analog-to-digital, digital-to-analog AT\&T Truevision Advance Raster Graphics Adapter 8 (TARGA 8) converted the analog signal originating from the videocamera into a digital signal before being processed by the microcomputer. Likewise, the digital signal coming from the microcomputer is converted into an analog signal when a frame is displayed on the image monitor (11).
3. Image monitor: A Sony color TV displayed live images from the videocamera and stored images from the microcomputer.
4. Microcomputer: An interface board was installed in an IBM PC AT to grab, store, analyze, and display digitized images obtained from the videocamera.
5. Thermal printer: A Shinko CHC-345 produces hard copies of images stored in memory.

## Video Image Acquisition

Video images were recorded in June 1989 from the lobby passageway of the Hall Building at Concordia University in Montreal, Quebec, Canada. Temperature was about $20^{\circ} \mathrm{C}$ $\left(68^{\circ} \mathrm{F}\right)$ and lighting was a mixture of natural and artificial. The videocamera was placed 6 m above the passageway and covered a floor area of approximately $4 \times 4 \mathrm{~m}$. The camera was also positioned in such a way that pedestrians were either coming toward or going away from the camera. In order to reduce pedestrian overlapping, the angle between the filming direction and a vertical line was set to approximately 25 degrees.
Because an image is composed of 65,536 pixels with 256 grey tones, the computational effort required by the microcomputer is considerable. In order to reduce this effort, a grid (i.e., a pattern of lines forming squares of uniform sizes) made of adhesive tape was laid on the floor of the passageway to permit the conversion of multiple-grey-level images (i.e., images of 256 grey levels) into bilevel images. The color of the adhesive tape was selected to clearly contrast with the color of the background. White tape was chosen for the grid with a width of 2 cm ( 0.8 in .).

Four different square sizes were experimented with: $30 \times$ $30 \mathrm{~cm}, 20 \times 20 \mathrm{~cm}, 10 \times 10 \mathrm{~cm}$, and $5 \times 5 \mathrm{~cm}$. With


FIGURE 1 Structure of image device system.
decreasing square size, the accuracy of results increases as does the computational effort. As a result, a compromise between accuracy and computational effort was made-a grid composed of $10-\times 10-\mathrm{cm}(3.9-\times 3.9-\mathrm{in}$.) squares was selected.

## ALGORITHM

Figure 2 shows the structure of the algorithm, which consists of eight steps. These eight steps along with a simple example are described in detail in the following discussion.

## Step 1. Conversion of Video Images

In this step, video images displaying pedestrian flow are converted into a discrete form of frozen frames. Frozen frames are two-dimensional arrays of images taken at contiguous time instants spaced at a regular time interval. Approximately three frozen frames are captured every second from the videotapes. Thus, image analysis of pedestrian movement will be performed by processing these frozen frames.

Figure 3 shows the simple example of three contiguous frozen frames. In this figure, a pedestrian is walking across the observation surface over which a white grid has previously been laid. The man shown in these frames is walking toward the camera.

## Step 2. Digitization of Frozen Frames

Using the TARGA 8 board, the frozen frames obtained from Step 1 were converted into two-dimensional arrays of $256 \times$ 256 pixels. Each frame is composed of a total of 65,536 pixels. Grey values for each pixel range from 0 to 255 , providing 256 shades of grey varying from black to white.

Figure 4 shows the printout of grey values of the pixels for the left leg of the pedestrian shown in the second frame of

Figure 3. In Figure 4, the grey values above 170 are underlined. The pixels of the white grid lines are assigned a high grey value, i.e., 170 and above, whereas the pixels of the black pants are assigned a low grey value, i.e., 100 and less.


FIGURE 3 Example of three contiguous frozen frames.


FIGURE 2 Structure of algorithm.

## Column Number




FIGURE 4 Grey values of left leg of pedestrian shown in second frame of Figure 3.

Step 3. Conversion of Images of 256 Grey Levels into Bilevel Images

One of the major problems encountered in processing image sequences is to extract useful information from images defined by 256 grey levels with a complex background. Much work is required from a microcomputer to process and analyze all pixels of an image of 256 grey levels. In order to reduce the required computer time to a minimum, the images of 256 grey levels are converted into bilevel images. Pixels of a bilevel image have either of two values- 0 or 1 . In this scheme, a pixel with a grey value of 0 is interpreted as a white point and a pixel with a grey value of 1 is interpreted as a black point.

A threshold range for a grey value of 1 is predetermined by visually analyzing the range of grey values of pixels belonging to grid lines in the images of 256 grey levels. For instance, a threshold ranging from 170 to 255 was selected. Thus, pixels within that range were converted into 1 ; otherwise, they were converted into 0 .

Figure 5 shows the three bilevel images obtained after the conversion of the three frozen frames shown in Figure 3. In Figure 5, all pixels whose grey values were outside the threshold range in the images of 256 grey levels were converted into white points. These pixels belong to the floor, pants, face, lower body, and most of the upper body of the pedestrian. On the other hand, all pixels whose grey values were within the range were converted into black points. These pixels belong to the grid lines, the shoulder region of the pedestrian, a part of the jacket carried over the shoulder, and the bald portion of the pedestrian's head.

## Step 4. Extraction of Rough Sketch of Pedestrian

The purpose of this step is to extract rough sketches of pedestrians from bilevel images. A reference image can be defined as a bilevel image containing stationary components only. The reference image contains the grid lines alone, as shown in


FIGURE 5 Example of bilevel images.

Figure 6. Therefore, rough sketches of pedestrians can be obtained by subtracting the bilevel images with pedestrians from the reference image.
Images containing rough sketches of pedestrians are called difference images. Let $G_{D}(x, y), G_{P}(x, y)$, and $G_{R}(x, y)$ denote the grey value of the pixel $(x, y)$ in the difference image, the image with pedestrians, and the reference image, respectively, where $(x, y)$ is the coordinate of the pixel in the image. Thus, $G_{D}(x, y)$ can be calculated as follows:
$G_{D}(x, y)=\left|G_{P}(x, y)-G_{R}(x, y)\right|$
In Equation 1, the values of $G_{D}(x, y), G_{P}(x, y)$, and $G_{R}(x, y)$ are either 0 or 1 . Hence, $G_{D}(x, y)=0$ when $G_{P}(x, y)=G_{R}(x, y)$. Otherwise, $G_{D}(x, y)=1$.

Figure 7 shows a difference image that contains a rough sketch of the pedestrian. This difference image was obtained


FIGURE 6 Reference image.


FIGURE 7 Difference image containing rough sketch of pedestrian.
by subtracting the second bilevel image shown in Figure 5 from the reference image shown in Figure 6. Furthermore, the difference image contains grid line noise that was induced by distortions originating from two sources, i.e., camera optics and recording system, and variations of light and weather. Line noise was also introduced by inaccurate differentiation of the two images.

From Figure 5, a pedestrian image may contain both white and black parts in a bilevel image. Hence, two different cases encountered during the subtraction process are schematically shown in Figure 8. These two cases are

1. The subtraction process for a white object yields a black cross in the difference image.
2. The subtraction process for a black object yields a white cross in the difference image.

Thus, Figure 8 shows that pedestrian shape remains in the difference image after subtraction even though the pedestrian may contain both white and black objects in the image.

Case 1:


FIGURE 8 Subtraction process.

## Step 5. Removal of Line Noise

This step aims to remove most of the line noise with a thinning procedure that uses a four-pixel scanning window. Figure 9 shows the scanning window that contains the scanning pixel $(x, y)$ itself and three neighbor pixels. Let $G_{D}(x+1, y)$, $G_{D}(x, y+1)$, and $G_{D}(x+1, y+1)$ denote the grey values of the three neighbor pixels $(x+1, y),(x, y+1)$, and $(x+1, y+1)$, respectively. Let $G_{t}(x, y)$ be the recalculated grey value of $G_{D}(x, y)$ obtained by using the thinning procedure. This procedure scans every pixel through the four-pixel window using the following rules:

1. If $G_{D}(x, y)=0$, then $G_{t}(x, y)=0$; and,
2. If $G_{D}(x, y)=1$, then
(a) if $G_{D}(x+1, y)=G_{D}(x, y+1)=G_{D}(x+1, y+1)=1$, then $G_{t}(x, y)=1$; (b) otherwise, $G_{t}(x, y)=0$. In other words, the thinning procedure removes a black pixel from the image if at least one of its three neighbors is a white pixel.

Figure 10 shows the result of removing line noise from the difference image shown in Figure 7. Most of the line noise


FIGURE 9 Four-pixel scanning window used to remove line noise.


FIGURE 10 Result of removing line noise from image shown in Figure 7.
has been eliminated. Also, the grid lines that constitute the rough sketch of the pedestrian become thinner than those shown in Figure 7. Thus, Figure 10 clearly shows the shape of the pedestrian accompanied by some remaining noise.

## Step 6. Reconstruction of the Shape of the Pedestrian

The purpose of this step is to further delete the remaining noise and to reconstruct the shape of the pedestrian simultaneously. A filling procedure including two substeps was developed for this step. The first substep finds the feature points in the rough sketch image. The second substep fills a certain region surrounding these feature points with black pixels.

Grid line segments in the image have a length of 6 to 7 pixels. Hence, as shown in Figure 11, a $7-\times 7$-pixel window (with a total of 49 pixels) was created for the filling procedure. As denoted in Step 5, $G_{r}(x, y)$ is the grey value of the scanning pixel $(x, y)$. The filling procedure is composed of the following two substeps.

## - Detection of Feature Points.

1. If $G_{t}(x, y)=0$, then pixel $(x, y)$ is not a feature point; go to the next pixel;
2. If $G_{t}(x, y)=1$, then, (a) If $G_{d}(x, y-2)=G_{t}(x, y-1)=$ $G_{t}(x, y+1)=G_{r}(x, y+2)=G_{t}(x-2, y)=G_{t}(x-1, y)=$ $G_{t}(x+1, y)=G_{t}(x+2, y)=1$, then pixel $(x, y)$ is a feature point that is stored in the computer memory; (b) otherwise, pixel $(x, y)$ is not a feature point. Go to the next pixel.

- Rebuilding of the Pedestrian Shape. The feature pixels detected in the previous substep are used to construct a new image. First, the feature pixels are placed in the new image. Then, for every feature pixel $(x, y)$ in the new image, a $7-\times$ 7 -pixel window filled with black points is positioned with its center at coordinates $(x, y)$. Thus, the new image is composed of a number of black squares.


FIGURE 11 Scanning window of filling procedure.

Figure 12 shows the new image obtained by the filling procedure. In this figure, the general shape of the pedestrian is represented by black squares whose size is determined by the grid size. The legs, upper body, and the jacket carried over the shoulder are clearly visible, but the representation of the pedestrian is coarse. In fact, the representation of pedestrian shape is directly affected by grid size.

## Step 7. Measurement of the Number of Pedestrians

A pedestrian-shape image may contain several black objects. As shown in Figure 12, one black object is a group of adjacent small black squares. Also, one black object may include more than one pedestrian because of overlapping. About 40 pedestrian-shaped images were randomly chosen to calculate the average size of a pedestrian in a black object. The sample included only adults of various types (e.g., fat, skinny, tall, and short). Results indicate that the average size of a pedestrian is approximately 1,500 black pixels. However, the average size of a pedestrian is also affected by camera position. Thus, the number of pedestrians in object $i$, if there are $k$ objects, can be calculated as
$p_{i}=\operatorname{Int}\left[T_{i} / 1,500\right] \quad i=1, \ldots, k$
where $T_{i}$ is the number of black pixels in object $i$, and $p_{i}$ is the number of pedestrians in object $i$ ( $p_{i}$ is rounded to the nearest integer). The total number $P$ of pedestrians in one image can be calculated as
$P=\sum_{i=1}^{k} p_{i}$

The knowledge of the number of pedestrians present in one image makes possible the determination of density. As previously defined, density is the concentration of pedestrians within a walkway. Because the grid or survey area is fixed, pedestrian density of the area can be calculated as
$D=P / A$
where $D$ is the density of pedestrians within the survey area


FIGURE 12 Reconstructed shape of pedestrian using filling procedure.
(number of pedestrians per square meter), and $A$ is the surface area of survey area $\left(\mathrm{m}^{2}\right)$.

## Step 8. Determination of the Direction of Pedestrian Movement

The purpose of this step is to determine the walking direction of the pedestrians in shape image $S_{1}$. Let $G_{S 1}(x, y)$ and $G_{S 2}(x, y)$ denote the grey values of the pixel $(x, y)$ in two contiguous shape images, $S_{1}$ and $S_{2}$, respectively. Shape image $S_{2}$ is obtained after shape image $S_{1}$. Also, let $G_{b}(x, y)$ be the grey value of the pixel $(x, y)$ of a new image that is obtained by performing the following Boolean-type operation: IF $G_{S_{1}}(x, y)$ AND $\left\{\mathrm{NOT}\left[G_{s 2}(x, y)\right]\right\}$ is TRUE, THEN $G_{b}(x, y)$ is TRUE, where $G_{s 1}(x, y), G_{S 2}(x, y)$, and $G_{b}(x, y)$ are TRUE if they have a value of 1 and are FALSE if they have a value of 0 . This Boolean operation can be explained by checking the following two conditions:

1. $G_{b}(x, y)=1$ if $G_{S_{1}}(x, y)-G_{S 2}(x, y)=1$;
2. Otherwise, $G_{b}(x, y)=0$.

The new image generated by the Boolean operation is called a direction image. The Boolean operation is different from the subtraction procedure that was discussed in Step 4. According to the Boolean operation, a black pixel $(x, y)$ is generated in the direction image only when both its corresponding pixel in image $S_{1}$ is black and its corresponding pixel in the image $S_{2}$ is white. Figure 13 shows the Boolean operation performed on two contiguous shape images. The direction image contains groups of black pixels, which are called direction objects. These direction objects represent pixels that were black in shape image $S_{1}$ and white in shape image $S_{2}$.

Pedestrians studied walked either in a northbound or southbound direction. Thus, the direction of movement of black object $i$, of $k$ objects, is determined by comparing the location of its direction object in the direction image with respect to its overall shape in image $S_{1}$. The topmost pixel of black object $i$ in shape image $S_{1}$ is first compared with the topmost pixel of its corresponding direction object in the direction image. If these two pixels have identical coordinates, then black object $i$ is moving in the southbound direction. Otherwise, the lowest


FIGURE 13 Example of determination of pedestrian direction.
pixel of black object $i$ in image $S_{1}$ is compared with the lowest pixel of the direction object. If they have identical coordinates, then black object $i$ is moving in the northbound direction. If none of these cases arises, black object $i$ in the shape image $S_{1}$ is not moving.

## DISCUSSION OF THE ALGORITHM AND RESULTS

## Complexity of the Algorithm

The time complexity of an algorithm can be defined as the total number of operations required to process input data and to produce output information when solving the problem. The big $O$ limit notation is used to describe the relationship between the time complexity and the size of the input data.

Let $n$ denote the total number of pixels in an image. In this case, $n=65,536$ pixels. The time complexity of the algorithm is

1. Total running time for Steps 1 and 2 is constant and is approximately 0.3 sec .
2. From Step 3 to Step 8, the algorithm includes conversion of images, extraction of rough sketch, thinning procedure, filling procedure, and determination of object size. The time complexity for each step is $O(n)$.

Therefore, the time complexity of the algorithm is $O(n)$. This relationship indicates that the upper bound of the computer time is a linear function of the size of the image. This feature also implies that the algorithm is efficient and powerful.

## Real-Time Analysis

Real-time analysis would be desirable for the application of this process. The real-time system requires that the response
time of the computer system be tied to the time scale of events occurring outside the computer. The computer must be able to process and output data within a critical specified time interval. This time interval can be determined by several factors such as the average walking speed of pedestrians, observation area of the camera, and processing capability of the algorithm. Computer time of about 0.5 sec or less to analyze an image is necessary to satisfy the real-time requirement.

For the current image system consisting of an IBM PC AT, a TARGA 8 board, etc., the computer time for processing and analyzing an image of 65,536 pixels is about 30 sec , which is much longer than the desired time of 0.5 sec .

The proposed algorithm is a linear function of the size of the image. Furthermore, operations in each step of the algorithm depend only on local information. In other words, input of one operation does not depend on the output of another operation. Thus, the entire operation in each step of the algorithm can be performed independently in parallel and very large scale integration (VLSI) architecture can be implemented to achieve the goal of real-time analysis.

Recent advances in VLSI technology have produced a strong impact on computer architectures and have created a new horizon for the implementation of parallel algorithms on hardware chips (12). Many books and articles have been devoted to VLSI algorithms and architecture and address implementation of image-processing algorithms that are particularly time-consuming and demanding of memory storage.

A study of implementing the VLSI architecture for the proposed algorithm is already in progress. Figure 14 shows the mesh-connected arrays for the thinning and filling procedures of the algorithm. Therefore, the real-time analysis should be attainable in the near future.

## Accuracy of the Algorithm

A computer program has been developed for Steps 3 through 8 of the algorithm. This program was written in PASCAL language. As described previously, scenes of people walking


PE ----Processing Element
FIGURE 14 VLSI architectures.
across the observation area were recorded on videotapes for about $11 / 2 \mathrm{hr}$. In order to examine the accuracy of the proposed algorithm, about 120 frozen frames containing one or more pedestrians were taken from the videotape. Using the TARGA 8 board, these 120 frozen frames were digitized into images of 256 grey levels. These images were then processed by the developed computer program, i.e., the objects in the images were extracted and analyzed to determine the number of pedestrians and their walking direction.

As many as eight pedestrians were visible in the images that were used to test the accuracy of the algorithm. Results obtained by the computer were compared with those obtained by visual counting on the image monitor. The comparisons show that the accuracy was about 100 percent for the images without any overlapped pedestrians. Overlapped pedestrians can be seen on the shape images in which some black objects contain more than one pedestrian. Overlapping occurs when pedestrians are walking abreast, when they are closely following one another, or when they are closely passing one another. For the case of an image in which each black object contains only one pedestrian, the algorithm is able to count the number of pedestrians perfectly. However, when the number of overlapped pedestrians and the degree of overlapping increases, the accuracy of measurement decreases. The overall accuracy for measuring the number of pedestrians in an image was about 93 percent for low- to average-density traffic situations.

The same 120 images were used to examine the accuracy of determining the walking directions of pedestrians. Pedestrian directions obtained from the computer program were compared with those obtained by visual measurement. Results of the comparisons indicate that the accuracy was about 100 percent for contiguous images in which no object merging or splitting was present. Object splitting occurs when a black object that contains two or more pedestrians in a shape image splits into two or more black objects in the next contiguous image. Object merging is the reverse situation. As the number of merging and splitting cases increases, the accuracy of the algorithm decreases. Overall accuracy of the algorithm for determining walking directions was over 92 percent for low- to average-density traffic situations.
In conclusion, results of the accuracy test indicate that this study has not yet reached the stage of implementation. In order to increase the accuracy of the measurement in the future, the vertical angle of the camera should be reduced to near zero. In other words, if the camera can be placed directly above the pedestrians, the occurrence of merging, splitting, and overlapping can be significantly reduced. Consequently, the average size (i.e., number of pixels) of pedestrians and their walking direction can be calculated more accurately.

## CONCLUSION

Traffic and transportation engineers continually require a more accurate and larger amount of pedestrian flow data for numerous purposes. Results indicate that automatic image analysis could prove valuable in a wide range of pedestrian data collection in the future that can accurately measure density and direction as well as speed and volume.

A new algorithm was developed to measure the number and walking direction of pedestrians. The algorithm consists of eight steps. An image device system was used to record pedestrian images in a hallway passage. Images were digitized using a TARGA 8 board and then converted into bilevel images. A thinning procedure was designed to remove the noise present in the images. Also, a filling procedure was used to reconstruct the shape of pedestrians. Number of pedestrians was obtained by measuring the number of black objects and their sizes in the image. Walking direction of pedestrians was determined by using a Boolean-type operation.
The results of complexity analysis show that the proposed algorithm is a linear function of the image size. When examining low- to average-density pedestrian flow situations only, the overall accuracy of the algorithm for measuring the number of pedestrians in an image was about 93 percent. Lowdensity situations occur at either level of service (LOS) A or B and the average-density situations occur at either LOS C or D , as specified in the HCM (7). The accuracy of determining the walking direction of the pedestrians was about 92 percent. Using the concept of parallel processing, real time analysis could be reached in the near future. Although still in the preliminary stages, this process is still incapable of measuring the pedestrian flow data under heavy pedestrian situations, but research of methods by which to overcome these limitations is already in progress.

In conclusion, results show that image analysis has significant potential in the area of automatic measurement of pedestrian flow data. Nevertheless, much effort will be required in the future to provide suitable software and hardware systems before reaching the stage of implementation.

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# Estimating and Updating Flows on Pedestrian Facilities in the Central Business District 

P. N. Seneviratne and M. Javid


#### Abstract

Pedestrian volume data are needed for a variety of purposeson the demand side, for examining trends and planning facilities, and on the supply side for evaluating safety and level of service, to name a few. However, lack of flexible analytical tools and data gathering costs limit authorities' ability to examine pedestrian flows and provide the required levels of service. Efforts to develop expansion models and demand models to minimize manual data gathering are also hindered by the lack of information about variation in flow over time, as well as uncertainty in the models. A statistical approach based on Bayesian theory is discussed that can combine available data, analysts' experience (subjective judgment), and short-period (sample) counts, to estimate the expected (mean) flow at a given site at a given time that will serve as input for a particular expansion model. Compared with sample means, Bayesian estimates are much closer to the true mean and, hence, expanded values would be less uncertain. Moreover, the technique could be used to update previous estimates by combining them with newly performed short-term counts.


As cities grow and traffic congestion in central areas becomes acute, planners and engineers will need to look more carefully at the possibility of exploiting the more efficient and practical modes of travel such as public transit and walking. In fact, several studies including Seneviratne and Morrall (1) and Bondada (2) have shown that, even at present, walking is the most frequently used mode for intra-central business district (CBD) trips. Unfortunately though, apart from isolated examples of aesthetically pleasing environments with priority for pedestrians, the concern in terms of long-term goals and objectives for this mode seems inadequate to encourage continued usage.

Some of the imbalances and inequities in investment can be attributed to the deficiency of information on pedestrian needs, network characteristics, and trip functions. How often are pedestrian volume counts taken on the network links, or how many cities conduct pedestrian origin-destination surveys? Nevertheless, pedestrian flow data are extremely important for purposes of planning and design. On the one hand, these data indicate levels of facility usage and trends that are needed for long-term facility planning and policy formulation. On the other hand, these data form the basis for identifying hazardous (high-conflict) sites, delays, and capacity problems.

Each municipality and provincial transportation agency in Canada has procedures for collecting and analyzing vehicular traffic data. However, pedestrian data are not collected or investigated with the same degree of intensity and enthusiasm.

[^13]Although alarming accident statistics and declining employment levels in CBDs have created a stronger need for analyzing pedestrian movement, the absence of flexible analytical tools, costs associated with manual collection, and lack of proven mechanical or electronic data collection techniques have forced authorities to shy away from the question of pedestrians.

Until the technology is fully developed and automated data gathering and analytical tools become easily accessible, local agencies could take advantage of several statistical techniques to minimize the cost of pedestrian data. A technique for estimating and updating short-term pedestrian flow or the input to expansion models is discussed. These input values are usually estimates of the mean (expected) flow during a short time interval of say 10 or 15 min . Sample data, experience of the analysts (subjective judgment), or historic data can be combined according to Bayesian statistical rules to obtain estimates of mean flow closer to the true mean.

## ESTIMATION OF PEDESTRIAN FLOW

Following the vehicular traffic flow theories postulated by Greenshields (3) and Greenberg (4), researchers of pedestrian movement such as Fruin (5), Navin and Wheeler (6), and O'Flaherty and Parkinson (7) concentrated mainly on evaluating walkway capacities and levels of service. Much of the work on the demand side involved development of multiple regression-type predictive models $(8-10)$ based on land-use variables at the specific sites. However, as pointed out by Davis et al. (11), transferability and long-term validity of these models are not clearly demonstrated. Davis et al. (11) proposed expansion models for crosswalk volumes based on data collected at eight sites in Washington, D.C. These authors found that hourly pedestrian volumes can be predicted from counts as small as 5 min during the middle of the hour being considered. From Seneviratne et al. (12), relationships also exist between hourly or daily pedestrian volumes and shortperiod counts. However, the strength and validity of the relationship between the two variables will be (a) dependent on the variability of volume and other characteristics of the sample sites, and (b) influenced by the position and length of the short-term count interval.
Thus, a certain degree of uncertainty can be expected in the validity of the expansion models, primarily in relation to the form and parameters of the models. For instance, Davis et al. (11) obtained logarithmic relations between short-period
counts and volumes for periods of up to 4 hr . However, these expansion models would only be valid for sites with a certain range of volumes because the sample sites were selected according to volume. Moreover, the models are unable to explain 100 percent of the variation and in some instances contain large standard errors of estimate. Thus, uncertainty results from the form of the model. Uncertainty in model parameters, on the other hand, stems from insufficient data. In other words, regression constants and coefficients contain uncertainty because they are estimated from small samples.

Uncertainty also stems from the short-term pedestrian flow itself. For instance, even with a sufficiently large data base, the fundamental uncertainty in pedestrian trip generation rates affects the validity of the estimates. Although fundamental or inherent uncertainty will remain despite sample size, the Bayesian approach can be used in two fashions to minimize some other sources of uncertainty in the estimates from expansion models. First, the Bayesian approach could be used to modify the chosen input value or the short-term (say 10min) count to bring it closer to the true mean of the $10-\mathrm{min}$ counts during the period under investigation. Alternately, if the underlying data base of the expansion models is available, modification of the expanded short-term counts can be done using the Bayesian approach.

In order to use the Bayesian approach, variations in shortterm counts and the underlying probability distributions need to be considered.

## DATA

Data from two surveys are used in the discussion and a numerical example given later. The first survey was an extensive pedestrian survey conducted in Calgary, Alberta, in 1983. As a facility usage survey, flows on the entire above-ground and at-grade networks were monitored for 6 hr over two consecutive weeks. Of the 32 midblock sites, flows in 30 were recorded at $5-\mathrm{min}$ intervals for 6 hr (i.e., during 0730 to 0900 , 1000 to 1130,1230 to 1330,1430 to 1530 , and 1600 to 1700 hr ). Two sites were used as control sites where the flows were monitored continuously from 0700 to 1900 hr for the entire 2 -week period. This large data base enabled analysis of variation in flow over time, as well as the development of expansion models.

The second survey was a limited survey conducted in the CBD of Montreal, Quebec, in 1989. This survey was confined to the noon-hour peak ( 1200 to 1300 hr ) when the pedestrian flows at 10 sites similar in terms of land use and volumes were monitored for 2 weeks.

## VARIATION OF PEDESTRIAN FLOW

Because of greater interaction between pedestrians, who have a tendency of walking side by side in pairs or groups, as well as trip generation rates that are influenced by capacities of transportation modes, elevators and escalators, and traffic signals, flows are highly irregular. Studies by Haynes (13), Seneviratne and Morrall (14), and Pushkarev and Zupan (9) have revealed that flows during small intervals are virtually irreproducible from one day to another. Thus, because the
input to an expansion model [i.e., the mean (expected) shortterm flow] is generally estimated from a few counts taken during the time period under investigation and the standard error is large, the expanded values may be uncertain.

Haynes (13) has shown that the standard error of estimate can be minimized by selecting short-term counts taken over intervals greater than 10 min . On the other hand, sample data could be combined with subjective judgment on the basis of experience and historical data to obtain the maximum likelihood estimate of the mean short-term flow. This approach is known as the Bayesian estimation technique.
The procedure for combining data depends entirely on the probability distribution of the flow at the investigated site, which has been found to follow a certain regular pattern during sufficiently large time intervals. For instance, even though short-term fluctuations are virtually unavoidable and of less significance to planners and engineers, Haynes (1.3) found that at midblock locations, the 1 -min flow is a normal random variable. Seneviratne and Morrall (14) show that the $15-\mathrm{min}$ flow during a.m., noon, and p.m. peaks could be represented by normal distributions.

After fitting three different distributions (i.e., normal, Poisson, and negative binomial), Javid (15) found that the 5 -min flow at intersections can also be reasonably approximated by a normal distribution as shown in Figure 1. Moreover, Javid (15) and Seneviratne and Morrall (14) have shown that the means of the 5 -min flow during a given hour at similar intersections as well as the means of the $15-\mathrm{min}$ flow at comparable (in terms of variation in flow) midblock sites are normally distributed. In other words, the short-term flow at each sample site $i$ is a normal random variable with mean $m_{i}$ and variance $s_{i}^{2}$ and the distribution of $m_{i}$ taken over all sites ( $i=1, \ldots, n$ ) is also normal in form with mean $m^{\prime}$ and standard deviation $\sigma^{\prime}$. This satisfies some of the prerequisites for using Bayesian updating (i.e., normal conjugate prior).

## BAYESIAN ESTIMATES OF EXPECTED FLOW

The Bayesian approach has had many applications in geotechnical engineering (16) in which testing is costly and the design characteristic (soil stability) is highly variable as are pedestrian and traffic flows. Yet, this approach has been used to a lesser extent in transportation planning (17).

Described in terms of notations, the analyst's objective should be to refine the mean (expected) flow $m_{s}$ obtained from a few short-term counts at a site $j$, which is similar to some sites for which sufficient data are available. Suppose that using experience and available data, $m^{\prime}$ or $\sigma^{\prime}$ can be estimated. Then, these two parameters can be combined with $m_{s}$ and $s_{s}$ to derive Bayesian estimates of the mean ( $m^{\prime \prime}$ ) and the variance ( $\sigma^{\prime \prime 2}$ ) of short-term flow at site $j$ and these estimates would be closer to the true short-term mean $m_{j}$.

Thus, the basic procedure consists of three simple steps. The first step involves finding or assuming the prior distribution of mean pedestrian flow per unit of time during the period of analysis. The prior distribution is essentially the distribution of the means of flow at several similar sites, and assumed to be the probability distribution of flow at the site under investigation. The form and parameters of the prior distribution are estimated from available data or analyst's experience with


FIGURE 1 Distribution of 5 -min flows.
similar sites. As mentioned previously, data from Montreal and Calgary suggest that the prior distribution is normal in form and parameters $m^{\prime}$ and $\sigma^{\prime}$ are obtained from
$m^{\prime}=\frac{1}{n} \sum_{\text {all } i} m_{i}$
$\sigma^{\prime 2}=\frac{1}{(n-1)} \sum_{\text {all } i}\left(m_{i}-m^{\prime}\right)^{2}$
The next step is to derive the sampling distribution and its parameters. The sampling distribution is the distribution of the few short-term counts at site $j$. Because the data base in this case is extremely small, an assumption can be made of the form of the distribution. If the distributions of flow at the sites included in the prior distribution are normal, then the distribution of the $k(k \ll n)$ short-term counts $X_{j}=$ $\left\{x_{1}, x_{2}, \ldots, x_{k}\right\}$ can be reasonably assumed to be normal. For example, Montreal and Calgary data were confirmed to have a normal distribution with $N\left(m_{j}, s_{j}\right)$ and estimates of $m_{j}$ and $s_{j}$ usually assumed to be equal to the sample parameters given by the following expressions:
$m_{s}=\frac{1}{k} \sum_{\text {all } k} x_{k}$
$s_{s}^{2}=\frac{1}{(k-1)} \sum_{\text {all } k}\left(x_{k}-m_{s}\right)^{2}$
To obtain the true mean of the $15-\mathrm{min}$ flow ( $m_{j}$ ) during the p.m. peak, ideally $m_{j}$ should be computed from several 15min observations at the same time over several days. However, because the objective is to minimize data collection
efforts, that $m_{j}$ equals $m_{s}$ can usually be assumed. The latter, $m_{s}$, could be the mean of 15 prorated 1-min counts or a randomly chosen $15-\mathrm{min}$ count during the p.m. peak. Thus, $m_{s}$ would often differ from $m_{j}$.

With the Bayesian approach, $m_{s}$ could be updated or refined to derive an estimate closer to $m_{j}$. In other words, if the forms and parameters of the prior and sample distributions are known, Raiffa and Schlaifer (18) have shown that the posterior or updated distribution parameters can be derived using empirical Bayesian analysis. Compared with sample and prior distribution parameters, the updated parameters are shown to be closer to the true parameters.

When prior and sample distribution are normal in form, Howard and Raiffa (18) found that the following relationships hold between the posterior distribution parameters and the prior and sample parameters. Thus, the third step is to enter the appropriate parameters obtained from the first two steps into the following expressions:
$m^{\prime \prime}=\frac{m^{\prime} /\left(\sigma^{\prime}\right)^{2}+m_{s} /\left(s_{s}^{2} / k\right)}{1 /\left(\sigma^{\prime}\right)^{2}+1 /\left(s_{s}^{2} / k\right)}$
$\sigma^{\prime \prime}=\left[\frac{1}{1 /\left(\sigma^{\prime}\right)^{2}+1 /\left(s_{s}^{2} / k\right)}\right]^{1 / 2}$
where $k, m_{s}$, and $s_{s}$ are sample size, mean, and standard deviation, respectively, of the sample distribution at Site $j$. The parameters of the prior and posterior distributions are denoted by ( $m^{\prime}, \sigma^{\prime}$ ) and ( $m^{\prime \prime}, \sigma^{\prime \prime}$ ), respectively.

In order to examine the extent to which sample size and sampling interval will make $m^{\prime \prime}$ approach $m_{j}$, the 5-min flows during the noon hour at a test site in Montreal were analyzed. The mean 5 -min flow ( $m_{i}$ ) computed from the 12 counts during
the hour was 74 pedestrians per 5 min and the standard deviation was 11 pedestrians per 5 min . As presented in Table 1 and shown in Figure 2, regardless of the sampling interval and the sample size $m^{\prime \prime}$ is generally closer to $m_{j}$ than is $m_{s}$.

## NUMERICAL EXAMPLE

The means of 5 -min flows during the noon hour from 16 midblock locations in Calgary followed a normal distribution with a mean of 180 persons per 5 min and a standard deviation of 16 persons per 5 min . These two values, which are based on sixty $5-\mathrm{min}$ counts at each site, are the prior distribution parameters $m^{\prime}$ and $\sigma^{\prime}$ that serve as input to Equations 5 and 6.

Suppose an evaluation is needed of the noon-hour volume at a new, previously unsurveyed midblock site similar to the

16 sites used in the prior distribution analysis. This evaluation may be estimated from an expansion model of the form given by Davis et al. (11):

Average hourly volume $=19.91 \mathrm{~m}^{0.79}$
where $m$ is the expected middle $5-\mathrm{min}$ flow.
Suppose that because of resource constraints, counts (i.e., $x_{1}, x_{2}, x_{3}$ ) could only be obtained during three 5 -min intervals commencing at $12: 25 \mathrm{p} . \mathrm{m}$. Thus, if the conventional procedure is followed, $m$ would be computed from these three counts. However, if the Bayesian approach is followed, these sample counts can be modified to derive a better estimate of $m$. With the latter approach, the three counts enable the computation of the sample distribution parameters $m_{s}$ and $s_{s}$ from Equations 3 and 4, respectively. Thus, the fundamental

TABLE 1 COMPARISON OF SAMPLE AND POSTERIOR PARAMETERS

| data set | k (Sampling Interval) | $\mathrm{m}_{\mathrm{s}}$ | $\mathrm{s}_{\mathrm{S}}$ | $\mathrm{m}^{\prime \prime}$ | $\mathrm{o}^{\prime \prime}$ |
| :---: | :---: | :---: | :---: | :---: | :---: |
| 1 | $4(12: 00-12: 20 \mathrm{~h})$ | 77 | 12 | 75 | 5 |
| 2 | $4(12: 20-12: 40 \mathrm{~h})$ | 76 | 10 | 75 | 5 |
| 3 | $3(12: 00-12: 15 \mathrm{~h})$ | 69 | 11 | 71 | 5 |
| 4 | $3(12: 15-12: 30 \mathrm{~h})$ | 71 | 10 | 72 | 4 |
| 5 | $3(12: 30-12: 45 \mathrm{~h})$ | 79 | 12 | 76 | 4 |
| 6 | $2(12: 00-12: 10 \mathrm{~h})$ | 78 | 7 | 76 | 4 |
| 7 | $2(12: 10-12: 20 \mathrm{~h})$ | 64 | 10 | 68 | 5 |
| 8 | $2(12: 20-12: 30 \mathrm{~h})$ | 69 | 10 | 71 | 5 |



FIGURE 2 Comparison of sample and posterior parameters.
data are $m^{\prime}=180, \sigma^{\prime}=16, m_{s}=220, s_{s}=25$, and $k=3$ ( 5 -min counts).

Substituting these values in Equations 5 and 6 yields posterior parameters or the Bayesian estimates $m^{\prime \prime}=202$ and $\sigma^{\prime \prime}=11$.

When compared with $m_{s}$ and $s_{s}, m^{\prime \prime}$ and $\sigma^{\prime \prime}$ are closer to the true mean of 200 persons per 5 min and standard deviation of 13 persons per 5 min computed from eleven $5-\mathrm{min}$ counts between 1200 and 1300 hr . This result shows that the Bayesian estimates of the parameters are closer to the true (observed) parameters than the sample parameters and ideally $m=202$ should be used in Equation 7. In this example, the difference in the expanded value from Equation 7 when using $m=m_{s}=220$ is approximately 100 pedestrians per hour.

The alternate procedure requires a knowledge of the standard error of estimate of the expansion model and the mean and standard error of the hourly volumes used in the regression analysis. Accordingly, the latter data would be $m^{\prime}$ and $\sigma^{\prime}$, whereas the standard error of estimate ( $s_{e}$ ) of the model would be $s_{s}$. The value of $m_{s}$ would be the average hourly volume obtained from Equation 7 for a given short-term count, in this case 220.
Both procedures are simple and data requirements are minimized (i.e., requires only one $5-\mathrm{min}$ or one $15-\mathrm{min}$ count) while achieving higher accuracy. These procedures could also be used for updating information. For example, next year flow data may need to be updated for the same site. In this case, current posterior parameters will act as prior parameters, and next year another three 5 -min counts will need to be taken during the same interval to obtain sample parameters. The next set of posterior parameters (Bayesian estimates) obtained from Equations 5 and 6 will be even closer to the true values than this year's values.

Bayesian approach can also be used as a routine (annual) updating procedure for all sites. If last year's counts are available, the figures could be updated with the aid of a short-term sample by following the previous steps.

## CONCLUSION

Given that expansion models can usually explain only a part of the variation, estimation errors could multiply unless input values are accurate. This condition was illustrated in the previous numerical example in which the variability of the shortterm counts resulted in a difference in volume of over 100 pedestrians per hour. Even when expansion models are unavailable, Bayesian estimation procedure could be used to derive the required information. For instance, most municipalities may have pedestrian flow data collected during intersection traffic counts in previous years, or may have performed some ad hoc counts at different sites. These data can be combined with sample counts to derive updated flows needed for level-of-service analysis.
Thus, in summary, the empirical Bayesian approach can be used to estimate flows at new sites or update flows at previously surveyed sites. The basic steps to follow at a new site are as follows:

1. Compute, from data available for sites similar in terms of magnitude and variation of flow or assume according to
experience, the mean $\left(m^{\prime}\right)$ and standard deviation $\left(\sigma^{\prime}\right)$ of, for example, p.m. peak volume likely to exist at the site in question.
2. Perform a sample count over a short period and compute mean $\left(m_{s}\right)$ and standard deviation $\left(s_{s}\right)$.
3. Substitute these values in Equations 5 and 6 to obtain expected volume ( $m^{\prime \prime}$ ) and standard deviation ( $\sigma^{\prime \prime}$ ). This value of $m^{\prime \prime}$ should be used in the expansion model instead of $m_{s}$.

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# Non-Euclidean Metrics in Nonmotorized Transportation 

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An investigation into the use of simple, real-world, non-Euclidean metrics for pedestrian and bikeway planning is described. The nature of non-Euclidean geometry is described and general applications are presented. Sensitivity of mode choice with respect to time is explained. The concept of geometric delay is discussed. In addition, principles of non-Euclidean metrics are applied to three areas: taxicab geometry, efficiency of alternative network designs, and plastic space. Results indicate that the application of non-Euclidean metrics is a useful tool in planning humanpowered transportation facilities. Further areas of research that will enhance and broaden the use of non-Euclidean metrics are identified.

Engineers and planners are well acquainted with the Euclidean metric (or geometry) through their formal education. Although the Euclidean metric appears to provide a good method for measuring the natural world, non-Euclidean metrics can be a valid tool for understanding the artificial manmade world. In fact, non-Euclidean metrics can be more useful to traffic engineers and urban planners than are Euclidean metrics (1), because the man-made world consists of a variety of different geometric forms. The usual distance measure, which is generally assumed and taken for granted, is not really what human beings perceive when they traverse city blocks.

Use of a non-Euclidean metric for pedestrian and bikeway planning is investigated in a preliminary way. First, the nature of non-Euclidean metrics is described and compared with that of the Euclidean metric, including some applications. Second, sensitivity of mode choice with respect to time is explained. The validity of considering time as more realistic than distance is examined by considering the phennmenon of transportation gaps occurring in most transportation systems in advanced countries (2). Third, the concept of geometric delay in Euclidean and non-Euclidean systems is discussed. Finally, three typical applications of non-Euclidean metrics are describedtaxicab geometry, efficiency of alternative geometric networks, and the concept of plastic space (3).

## NON-EUCLIDEAN METRICS AND SPATIAL ARRANGEMENT

A metric space comprises a set of points and a positive, subadditive distance function that relates every pair of points. The Euclidean distance relation $d_{E}(i, j)$ is the most frequently used metric in engineering and urban planning. This distance is the length of the shortest possible path joining a pair of points
(i,j) and only one such path can exist for each pair of points. In two dimensions, the Euclidean metric is given by the expression
$d_{E}(i, j)=\left[\sum_{k=1}^{2}\left(x_{i k}-x_{j k}\right)^{2}\right]^{1 / 2}$
When the points are characterized in $m$ dimensions, Equation 1 becomes
$d_{E}(i, j)=\left[\sum_{k=1}^{m}\left(x_{i k}-x_{j k}\right)^{2}\right]^{1 / 2}$
When the points lie in a three-dimensional space, $m=3$. This Euclidean space also has special significance for humanpowered transportation because, if the third dimension is altitude, greater human effort may be expended in using it, for example, in going up grades.
The Euclidean metric is a special case $(p=2)$ of the Minkowski $p$ metrics defined in $m$-dimensional space by the relation
$d_{E}^{p}(i, j)=\left[\sum_{k=1}^{m}\left|x_{i k}-x_{j k}\right|^{\rho}\right]^{1 / p}$
where $p$ is any positive integer.
For $p=1$ and $m=2$, the distance
$d_{T}(i, j)=\left|x_{i 1}-x_{j 1}\right|+\left|x_{i 2}-x_{j 2}\right|$
(the sum of intervals along each axis) is of great interest in civil engineering (Figure 1). This two-dimensional metric is generally known as the taxicab, pedestrian, Manhattan, or city-block metric (3).
The taxicab metric (or pedestrian metric) is intuitively appealing to transportation engineers, urban planners, and infrastructure designers. Particularly important to professionals are questions connected with the optimum location of apartments, industrial complexes, phone booths, sidewalks, and other facilities. In fact, any problem connected with a grid pattern of streets can be solved in terms of the taxicab metric. Krause (1) has investigated several real-world problems with taxicab geometry.
Radial, circumferencial, triangular, hexagonal, octagonal, and $n$-directional street patterns also yield examples of specific metrics that are in use in the optimum location problem (3).
As shown in Figure 1, in two-dimensional Euclidean space the locus of all points of distance $R$ from the origin $O$ defines


FIGURE 1 Various Minkowski metrics.
a circle of radius $R$. However, if the taxicab metric is adopted the locus of points equidistant from $O$ describes square $A X Y Z$ of diagonal of length $2 R$. In the metric for $p=\infty$, another square $L M N Q$ whose sides are each $2 R$ can be defined that represents the locus of all points equidistant from $O$.
Although in Figure $1 O A$ appears to be longer than $O B$, in the taxicab metric $O B$ is longer than $O A$. In other words,
$d_{E}(O, A)>d_{E}(O, B)$
$d_{T}(O, A)<d_{T}(O, B)$
Also, given, for example, that $d_{p}(A, B)=6, d_{p}(B, C)=10$, and $d_{p}(A, C)=8$, the configurations of these distances will appear as shown in Figure 2 for the taxicab (left) and Euclidean (right) metrics, respectively. The differences between the two parts of the figure illustrate the fact that in the taxicab metric a pedestrian does not take the shortest route but fol-
lows a grid street pattern in going from one point to another.
In general, the spatial arrangement or locational pattern of objects (such as man-made structures) may be used to determine a convenient metric for the space.

## CONSTRAINTS IN HUMAN-POWERED TRANSPORTATION

If one critically observes the range of the transportation function, a hypothesis can be made that in practice three modes of transportation dominate the overall hierarchy of transportation available to people: walking for short distances, cars for medium distances, and airplanes for long distances. Transportation planners are well aware of the refusal distance of the average pedestrian, usually 400 m (or $1 / 4 \mathrm{mi}$ ). Beyond 400 m , the majority of pedestrians demand some kind of a mechanical device or system to transport them. For instance,


FIGURE 2 Taxicab and Euclidean metrics.
a pedestrian who needs to travel a distance of $4 \mathrm{~km}(2.5 \mathrm{mi})$ will not agree to spend 50 min walking. A faster means of transport will be sought. Ample evidence exists to show that the trip maker's choice of mode is not based on cost alone, but rather on travel time. Conceptually, distance is related to time (2), leading to the importance of delay (transportation delay or geometric delay), a topic discussed later.

Table 1 indicates that when the time of travel is doubled, distance covered increases tenfold, whereas speed of travel increases fivefold. This phenomenon, which was studied by Bouladon (2), generally produces the three dominant modes: walking, driving a car, and flying by plane. At the same time, the phenomenon produces pronounced transportation gaps as shown in Figure 3. Of course, these data are not uniform for all populations but depend on the aggregate level of economic development prevalent in the society under examination (2). In general, $t=6.6 d^{0.3}$ where $t=$ time (min) and $d$ $=$ distance $(\mathrm{km})$, or $t=7.6 d^{0.3}$ where $t=$ time $(\min )$ and $d$ $=$ distance (mi).

On the one hand, the notion of transportation gaps can be easily dismissed on the grounds that evidently there is no market available for a particular mode in the transportation hierarchy that results in a gap. On the other hand, it may prove most advantageous to understand the real needs of the trip maker and the boundary conditions that society and the built environment have imposed on different modes in this hierarchy.

Bouladon (2) has demonstrated that the spectrum of transportation modes can be divided into roughly five areas, as shown in Figure 3. When demand for transport (vertical axis)


FIGURE 3 Transportation gaps.
is plotted against the speed, time, or distance measure (horizontal axis), the transportation range is well covered by pedestrians, cars, and air transport. Every mode is competent and cfficient over a certain distance, time, and speed range, according to the technology and economics inherent in the design of the system under study. Because time is the sensitive variable, the challenge for the transportation engineer is to somehow reduce the time needed to walk or bicycle a discrete distance. Reductions in time can best be achieved through examining ways to improve pedestrian networks and decrease time delays for pedestrians and bicyclists. Note that among the various modes of transportation available, the pedestrian mode has the largest demand.

TABLE 1 GENERAL TRANSPORT CONCEPT: DISTANCE, TIME, AND SPEED RELATIONSHIPS

| Distance, $d$ |  | Time, $t$ uin | Theoretical transport spee.d |  | Transport alternative |
| :---: | :---: | :---: | :---: | :---: | :---: |
| mi | km |  | mph | km/h |  |
| 0.25 | 0.4 | 5 | 3 | 4.8 | Walking |
| 0.62 | 1 | 6.6 | 5.6 | 9.1 | Bus or Bicscle |
| 2.5 | $t$ | 10 | 15 | 24 | Subway/Light Rail |
| 6.2 | 10 | 13.2 | 28 | 15.5 | Car on City Street |
| 25 | 40 | 20 | 70 | 120 | Car on Erecway |
| 62 | 100 | 26.1 | $1+1$ | 228 | Train |
| 250 | 400 | 40.0 | 375 | 605 | Small Plane |
| 620 | 1000 | 52.5 | 706 | 11.40 | Jet |

$t=6.6 d^{(1) .3} \quad t=$ time in minutes and $d=$ distance in km .
$t=7.6 d^{(0.3} \quad t=$ time in minutes and $d=$ distance in miles.

## Geometric Delay

The concept and measurement of delay for motorized transport is well established, particularly with the adoption of the 1985 Highway Capacity Manual (4) although Webster (5) used this concept as early as 1966 in the context of delay created by traffic signals. Geometric delay, similar to signal delay, is connected with the geometrics of the layout of transportation facilities, such as that experienced while negotiating rotary intersections. Several researchers have measured geometric delays for vehicles on highway and street networks. However, little investigation has been carried out to measure the magnitude of geometric delay for pedestrians and bicyclists using city streets (6).

Geometric delay in the context of Euclidean and nonEuclidean distance needs to be examined by investigating the time taken by a pedestrian to traverse a distance from, say, $A$ to $J$ in a typical grid street network shown in Figure 4 where the city blocks are $L \mathrm{ft}$ square and the streets are $W \mathrm{ft}$ curb to curb. Three cases are considered.

- Case 1. If there were no buildings or traffic to obstruct the pedestrian, the shortest path between $A$ (origin) and $J$ (destination) would be the straight line connecting $A$ and $J$. If the blocks were 400 by 400 ft , and the streets 40 ft wide, the Euclidean distance $d_{E}$ would be
$d_{E}=\left[(2 L+W)^{2}+(3 L+3 W)^{2}\right]^{1 / 2}=1,565 \mathrm{ft}$
Assuming walking speed $=4 \mathrm{ft} / \mathrm{sec}$, Euclidean walking time $t_{E}=1,565 / 4=391 \mathrm{sec}$.
- Case 2. If the walking domain of a pedestrian consists of all the sidewalks adjacent to the buildings and also, if there is no vehicular traffic, street furniture, guardrails, or traffic control devices to obstruct movement, then the pedestrian can follow any path within the walking domain to reach the destination, to minimize walking distance $d_{G}$ and, naturally, walking time. This concept is referred to as the geometrical minimum walking time $\left(t_{G}\right)$. Referring to Figure 4,

$$
\begin{aligned}
d_{G} & =A B+B C+C E+E G+G I+I J \\
& =40+400+402+402+402+400 \\
& =2,046 \mathrm{ft}
\end{aligned}
$$

and assuming a walking speed of $4 \mathrm{ft} / \mathrm{sec}$,
$t_{G}=2,046 / 4=512 \mathrm{sec}$

- Case 3. However, if a pedestrian's freedom of movement is constrained by motor vehicles, street furniture, guardrails, and traffic control devices, the pedestrian would have to conform to traffic laws and follow the taxicab or non-Euclidean path as shown in Figure 4.

$$
\begin{aligned}
d_{T} & =A B+B C+C D+D E+E F+F G+G H+H I+I J \\
& =4 W+5 L \\
& =2,160 \mathrm{ft}
\end{aligned}
$$



FIGURE 4 Geometric delay for pedestrians.
and
$t_{T}=2,160 / 4=540 \mathrm{sec}$
In addition, this pedestrian would have to wait at four street crossings for the Walk signal of, say, $30 \sec (4 \times 30=120$ $\mathrm{sec})$. Thus, although $d_{T}=2,160, t_{T}=540+120=660 \mathrm{sec}$, which could be considered as the practical minimum walking time.

Thus, geometric delay is the practical minimum walking time (using legal paths) minus the geometrical minimum walking time previously defined. Summarizing the results of the three typical cases, $t_{E}=391 \mathrm{sec}$; $t_{G}=512 \mathrm{sec}$; and $t_{T}=660$ sec , in which $t_{G}$ and $t_{T}$ are 31 and 69 percent higher than $t_{E}$, respectively, and $t_{T}$ is 29 percent higher than $t_{G}$. These calculations amply demonstrate the fact that geometric delay represents a significant component of the pedestrian's travel time.

Naturally, these values will vary depending on the relative location of origins and destinations on the street network, size of blocks and streets, and pedestrian signal cycle lengths.

Therefore, a reasonable expectation would be for transportation engineers and urban planners to minimize pedes-
trian and bicyclist geometric delay from all known sources, or at least to be cognizant of geometric delay.

## APPLICATIONS

## Taxicab Geometry

Of the many applications of non-Euclidean metrics, the one that has been investigated most is taxicab geometry, which has special significance in city planning. The only assumption in taxicab metrics is that the street system be in the form of a square or rectangular grid.

In many cities around the world, particularly in China, India, and North America, the spatial pattern of streets is usually a grid and the use of taxicab geometry is applicable. Spatial separation is more realistic using the taxicab metric $d_{T}(i, j)$ rather than the Euclidean metric $d_{E}(i, j)$, and the following elementary applications of taxicab metric are useful:

1. Trade, catchment, or market areas can be demarcated on maps by replacing $d_{E}(i, j)$ by $d_{T}(i, j)$. Several examples of this application have been used in the past. For example, if two roommates having jobs at locations $A$ and $B$ decide to find an apartment such that the sum of the distance they have to walk to work is no more than 18 blocks, they could demarcate a taxicab ellipse as shown in Figure 5 and seek an apartment within its confines.
2. Facility location using the taxicab metric is also applicable. Figure 6 shows an example of facility $P$ whose location minimizes the distance $d_{T}(P, A)+d_{T}(P, B)+d_{T}(P, C)+$ $d_{T}(P, D)$. All points within the crosshatched area make this condition possible.
3. The combination of walking distances and ideal location of mass transit (or subway stations) is another interesting area of investigation. Here, of course, the possibility could exist that the subway system is not running parallel to the street system. Bus systems can also be investigated in a similar way.


FIGURE 5 Taxicab ellipse.


FIGURE 6 Taxicab metric.

## EFFICIENCY OF ALTERNATIVE NETWORK DESIGNS

As previously discussed, a close parallel exists between the geometrics of pedestrian movement on sidewalks and motor vehicles on streets. Certain forms of delay incurred by pedestrians are also similar to vehicular delays (6). In these comparisons, time-distance and cost-distance immediately come to mind. For pedestrians and bicyclists, the question of minimizing time-distance is paramount.

In street systems across the world, alternative plans range from radial, circumferencial, or grid patterns for traditional cities with high-density centers, to highly complex combinations for the modern expansions of existing cities. Naturally, different geometric patterns of streets are associated with different patterns of relative accessibility.

Holroyd and others $(7,8)$ have examined several different street systems as shown in Figure 7. These systems (consisting of circular cities with a 1 -mi radius) make use of internal or external ring roads whereas others rely on radial, rectangular, and other polygonal networks. Efficiency of these networks is evaluated for internal movement when both origin and destination are inside the city. Average walking or biking distance between random pairs of points (origins and destinations) is presented in Table 2. Also, Figure 8 shows network street length or cost versus average trip length. If the entire city were paved (hypothetically) and a pedestrian were able to move in Euclidean space with no obstructions, then trip length would be 0.905 mi for a city with a radius of 1 mi . This (direct) case cannot be represented by a point on Figure 8. Random trip origin and destination point pairs are the same in all eight cases.

A broad conclusion that can be deduced from this experiment (Figure 8) is that the best results are obtained by adopting a street system with a radial, rectangular (grid), or internal ring configuration, because such systems minimize the average trip length as well as the total length of the street network.

The search for efficient geometrical designs for transportation networks is long standing. Elevated pedestrian bridges and underground pedestrian tunnels (underpasses) together with skywalks, such as those provided at Cincinnati, Spokane,


TABLE 2 AVERAGE LENGTH OF TRIPS IN A CIRCULAR CITY

| Configuration | Trip Length | Trip Length <br> $a=1$ mile | Street Length <br> (miles) |
| :--- | :---: | :---: | :---: |
| 1. Direct | $0.905 a$ | 0.905 | $\infty$ |
| 2. Radial | $1.333 a$ | 1.333 | 24 |
| 3. Ext. Ring | $2.237 a$ | 2.237 | 30.28 |
| 4. Int. Ring | $\frac{4 a}{3}-\frac{(1-\pi) b}{2}+\frac{1 b^{2}}{3 a^{2}}$ | 1.445 | 27.14 |
| 5. Radial-arc | $1.104 a$ | $1.10-4$ | 42.84 |
| 6. Rectangular | $1.153 a$ | 1.153 | 28 |
| 7. Triangular | $0.998 a$ | 0.998 | 43 |
| 8. IIexagonal | $1.153 a$ | 1.153 | 45 |



FIGURE 8 Average trip length versus street length.

Calgary, and Minneapolis, serve to reduce walking distance and reduce accidents.

## CONCEPT AND USE OF PLASTIC SPACE

Several researchers in recent years, particularly Fores (9) and Ewing (10), have examined the relationships of time and distance in defining the use of plastic space. For pedestrians and bike users, the consideration of time is more realistic in space than just mileage.

Correspondence between time space and geographical space is examined for the Washington State University (WSU) campus. A matrix of travel times to 25 crucial points on the campus map of WSU was recorded by using the sidewalks and legal pedestrian crossings by the shortest path. A similar matrix of these same 25 points was recorded by driving on the campus, also by the shortest paths along the campus streets. Figures 9-11 represent the geographical space, pedestrian walking time space, and vehicular driving space of WSU campus, respectively. Transformations of space-stretching and space-


FIGURE 9 WSU campus in geographical space ( $\mathbf{1} \mathrm{in} .=1,000 \mathrm{ft}$ ).


FIGURE 10 WSU campus in pedestrian time space ( $0.5 \mathrm{in} .=1 \mathrm{~min}$ ).
shrinking because of the use of different modes is evident by comparing these three illustrations. The WSU campus is particularly efficient for the pedestrian mode as compared with the vehicular mode and because of the provision of several automobile-free zones.

The concept of plastic space has important applications in exploring the efficient forms of spatial reorganization. Plastic space also has the potential for use in policy in terms of decision making regarding future investments. For example, additions of streets and sidewalks to an existing network could be configured by comparing Euclidean and non-Euclidean metrics applied to a street system to find the impact of such an addition. Transportation planners may find this approach of use in studying the potential impact of technical and economic proposals. Weir (11) has applied this method at State College, Pennsylvania, with success.

Plastic space would be particularly useful in parts of the city where pedestrian movement is predominant for several good reasons, one of which may be to reduce pedestrian delay and possibly also to increase safety. Provision of suitable footbridges over streets, skywalks, and automobile-restricted zones may be considered. Designing streets and pedestrian walkways more sensitively is really what is called for.

## CONCLUSION AND RECOMMENDATIONS

Three aspects of human-powered transportation have been described-general nature of non-Euclidean metrics, sensitivity of the pedestrian and bicycle mode to travel time, and basic concepts of geometric delay. Some practical examples


FIGURE 11 WSU campus in vehicular time space ( 1.25 in . = 1 min ).
of applying these and other concepts to the human-powered mode have also been described.

Knowing the principles and laws governing human-powered transportation is not enough. One must be capable of applying these principles in planning for pedestrians and bicycle users in a world competing vigorously for time and space. From this standpoint, the ideas expressed are just the beginning of an ongoing research.

Areas of research stemming from this investigation include the following:

1. A study of geometric delay for pedestrians and bicycle users relative to different geometric networks and ways in which such delay can be minimized (in the same way that traffic engineers minimize delay for motor vehicles) needs to be undertaken;
2. A study of plastic space for designing and evaluating plans for pedestrian and bicycle users movement networks (facilities) particularly for minimizing walking and bicycleriding distances appears to be essential where nonmotorized transportation is significant;
3. The question of improving pedestrian and bikeway safety relative to the motor vehicle, through a thorough examination of street geometry, traffic control devices, skywalks, and underground passageways, should form part of any street expansion or alteration system; and
4. The science of facilities location for different geometric network configurations, particularly for intermodal efficiency, for example, applied to pedestrian and subway systems would prove beneficial.

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# Relationship Between Child Pedestrian Accidents and City Planning in Zarqa, Jordan 

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The relationship between child pedestrian accidents and city planning was studied in Zarqa, Jordan. Variables considered included road pattern, road density, population density, size of green areas, and number of schools in the area. Data were collected for child pedestrian accidents from police records. Analysis of these data revealed several conclusions among which is that children 5 to 9 years of age account for 33 percent of the total number of pedestrian casualties and 49 percent of child pedestrian casualties. This group constitutes about 17 percent of the total population of Jordan. The city was divided into 16 study zones and statistical analysis was performed using multiple regression techniques. Significant relationships were obtained between child pedestrian accidents and several land use variables. Developed models can be used to give an estimate of the reduction in child accidents because of changes in road pattern and other variables. From these models, a 25 percent reduction in the number of four-leg (cross) intersections could reduce accidents by approximately 24 percent.

Accidents in residential areas have long been recognized as a problem of major concern for the general public and road safety authorities. Profound and direct impacts on the community occur because the close-to-home occurrence of these accidents frequently involves children.

Road accidents within the young age group is of great importance in Jordan because children under 15 years constitute about 50 percent of the total population. In response to the high exposure of this age group to traffic accidents, an investigation was conducted to ascertain the relationship between child road accidents and city planning aspects that may explain the high incidence of casualties in this age group. In Jordan, 1981 statistics showed that children under 15 years account for 61 percent of total pedestrian casualties (1).

Whitelegg (2) states that an urban system carries an intrinsic level of risk and can be assessed in statistical terms. He indicated that the close association between geometric design and layout and accident probabilities is an important element in the exploration of spatial selectivity and accident causation.

Appleyard (3) reports findings related to spatial selectivity and child casualties. In California, more accidents involving children ( 55 percent) occurred between intersections than at intersections (32 percent). Backett and Johnson (4) found that child casualties were more likely to occur in neighborhoods with few back yards, few playgrounds, and low parental supervision. This finding is closely related to those of Blackman (5), who found that death rates were higher for pedestrians

[^14]in poor areas because reduced car ownership resulted in more walking. In addition, poorer levels of recreational facilities lead to more on-street activity in these areas.

A United Kingdom study of 9,000 streets in local authority residential estates (6) found that accident rates were low in cul-de-sacs and high in estates designed in accordance with traditional practice. Children represented 76 percent of the casualties and 49 percent of the fatalities. The mean pedestrian accident rate in simple cul-de-sacs was less than onesixth of the overall mean.

Whitelegg (2) reports findings of a study in Gothenburg, Sweden, where most accidents involving children occurred in residential areas, but accident rates differed by district. Older areas had six times the accident risk of newer areas in which pedestrian and vehicle routes are separated.

One study that investigated the relationship between pedestrian accidents (not just children) and city planning was that of Crompton (7). Pedestrian casualties were studied in 474 $1-\mathrm{km}$ squares distributed in regions of England and Wales. All squares with less than 20 percent of the area developed and all predominantly nonresidential squares were omitted. Multiple regression analyses were carried out with pedestrian casualty rates per square kilometer of development as the dependent variable and with three different sets of independent variables: activity set (including numbers of pedestrians, vehicles, and parked vehicles), census data (including population, number of households, etc.), and land use set (including road length, number of shops, number of junctions, etc.). Crompton (7) concluded that land use variables influence pedestrian casualty rates to a considerable extent.

## STUDY OBJECTIVES

An attempt will be made to define and quantify the problem of child traffic safety in selected Jordanian cities. The relationship between city planning criteria and child road accidents was investigated with the aim of classifying factors that make the highest contribution to the problem of child accidents. The following is a list of the main objectives of this research:

- Quantify the problem of child traffic safety in Jordanian cities using the city of Zarqa as a study case,
- Analyze the spatial distribution of child road accidents in Zarqa,
- Establish the relationship between child accidents and several city planning criteria, and
- Suggest guidelines for improving traffic safety in existing residential areas as well as new areas.

The city of Zarqa was chosen because of its relatively large population. Data, which included population density, road patterns, and location of schools and recreational facilities, were also available on town planning criteria in different zones. Also, in Zarqa, the difference between old and new areas can be noticed clearly.

## STUDY AREA AND DATA COLLECTION

Zarqa is the second largest city in Jordan, with a 1988 population of nearly 350,000 . The city is located 30 km northeast of Jordan's capital, Amman. Figure 1 shows the road network in the city. The grid system of road pattern is dominant in Zarqa's central areas, whereas in modern areas in the northern part of the city, curvilinear patterns with loops and cul-de-sacs are the main feature of the network.
In order to analyze the spatial distribution of child pedestrian accidents, the city was divided into 16 study zones cor-


FIGURE 1 Zarqa road network.
responding to administrative municipal divisions as well as zones used in the 1979 census. These zones are shown in Figure 2.

Information on child pedestrian accidents was obtained from files in police headquarters in Amman for 1988. Data were limited to 1988 because of a new system of accident recording implemented in 1987. In this system, information on age, sex, and pedestrian action was available as well as the approximate location of the accidents. Child pedestrian accidents were allocated to the zones shown in Figure 2.

One year of data was considered sufficient because the analysis focused on the spatial distribution of child road accidents. An analysis of the trend of child road accidents was not under consideration. Such a trend will have little or no influence on the spatial distribution of child road accidents.

Information on road density and pattern was obtained from city maps. The area of roads in each zone was calculated as well as the number of four-leg (cross) intersections and their percentage.
Data on population density for each zone were obtained from the Department of Statistics. Data about elementary schools, parks, and playgrounds were obtained from the city government. These data are presented in Table 1.

## CHILD PEDESTRIAN CASUALTIES

Figure 3 shows the distribution by age group of pedestrian accidents in Zarqa. From Figure 3, the group of children 5 to 9 years old has the highest incidence of road accidents. As


SCALE 1-10,000
FIGURE 2 Zarqa study zones.

TABLE 1 DATA FOR TOWN PLANNING CRITERIA IN ZARQA

| Zone | Area of Zone ( $\mathrm{km}^{2}$ ) | Area of Streets ( $\mathrm{km}^{2}$ ) | Road Density (\%) | Population (1988) |  | No. of Intersections |  | No. of Elementary Schools | Green Area (donums) |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: |
|  |  |  |  | Per Zone | Density (persons/donum) | Four Legs | Total |  |  |
| 1 | 0.456 | 0.158 | 34.85 | 19.036 | 41.746 | 22 | 41 | 58 | 0.000 |
| 2 | 1.228 | 0.384 | 31.15 | 18.662 | 15.197 | 34 | 72 | 19 | 8.466 |
| 3 | 2.037 | 0.490 | 24.06 | 70.117 | 34.422 | 51 | 105 | 30 | 0.000 |
| 4 | 0.875 | 0.119 | 13.53 | 1.509 | 01.724 | 7 | 32 | 2 | 5.163 |
| 5 | 0.684 | 0.170 | 24.84 | 26.049 | 38.003 | 43 | 55 | 28 | 0.000 |
| 6 | 0.641 | 0.150 | 23.30 | 22.480 | 35.070 | 27 | 42 | 16 | 9.531 |
| 7 | 0.759 | 0.203 | 26.71 | 21.923 | 28.971 | 14 | 47 | 10 | 0.000 |
| 8 | 1.345 | 0.417 | 31.01 | 2.091 | 01.555 | 26 | 74 | 11 | 9.281 |
| 9 | 0.743 | 0.249 | 33.47 | 29.862 | 40.101 | 10 | 38 | 7 | 0.000 |
| 10 | 2.592 | 0.400 | 15.74 | 43.409 | 15.324 | 18 | 70 | 27 | 0.000 |
| 11 | 0.634 | 0.128 | 18.39 | 10.645 | 15.339 | 15 | 40 | 6 | 0.000 |
| 12 | 2.195 | 0.669 | 30.47 | 893.000 | 00.407 | 13 | 91 | 7 | 41.475 |
| 13 | 1.221 | 0.289 | 23.66 | 11.810 | 09.672 | 19 | 47 | 7 | 0.000 |
| 14 | 1.114 | 0.274 | 24.55 | 22.480 | 20.180 | 19 | 56 | 22 | 0.000 |
| 15 | 1.193 | 0.237 | 19.84 | 10.056 | 08.425 | 14 | 62 | 6 | 0.000 |
| 16 | 0.892 | 0.177 | 19.77 | 5.198 | 05.227 | 2 | 12 | 5 | 0.000 |

Note: 1 donum $=1000 \mathrm{~m}^{2}$.


FIGURE 3 Pedestrian casualties in Zarqa.
might be expected, boys outnumbered girls in road accidents with the ratio of girls to boys roughly 1 to 3 . The 5 to 9 year age group accounts for 49 percent of child pedestrian casualties but constitutes about 34 percent of the child population.

Nearly 58 percent of child pedestrian accidents occurred between intersections (midblock). About 79 percent of pedestrian accidents involving children occur while crossing the street, 8 percent occur while crossing in front of parked cars, and 9 percent while playing or standing in the street. Detailed numbers of these casualties by age group are shown in Table 2.

## MULTIVARIATE MODELS OF CHILD PEDESTRIAN CASUALTIES

Multiple regression analyses were carried out with child pedestrian casualties as the dependent variable and with several independent variables representing city planning criteria in each of the 16 zones. The two best models are as follows:

$$
\begin{align*}
& \text { Model } 1\left(n=16, R^{2}=0.82, F=19.02\right) \text { : } \\
& \begin{aligned}
1 / \ln (\mathrm{CP})= & 0.97-0.05 \ln (\mathrm{PD}) \\
& +0.26 / \ln (\mathrm{SR})-0.18 \ln (\mathrm{CI})
\end{aligned}
\end{align*}
$$

TABLE 2 PEDESTRIAN BEHAVIOR AT THE TIME OF THEIR INVOLVEMENT IN AN ACCIDENT IN ZARQA

| Age Group (years) | No. of Pedestrians by Behavior |  |  |  |  |
| :---: | :---: | :---: | :---: | :---: | :---: |
|  | Crossing Street | Crossing in Front of Parked Cars | Riding <br> a Bike | Playing or Standing on Street | Not Specified |
| 1-2 | 18 | 4 | 0 | 5 | 0 |
| 3-4 | 97 | 11 | 1 | 1 | 0 |
| 5-6 | 106 | 14 | 0 | 9 | 3 |
| 7-8 | 55 | 2 | 0 | 7 | 2 |
| 9-10 | 46 | 5 | 0 | 9 | 2 |
| 11-12 | 30 | 2 | 2 | 8 | 3 |
| 13-14 | 22 | 1 | 3 | 4 | 1 |
| Total | 370 | $\overline{39}$ | $\overline{6}$ | $\overline{43}$ | 11 |
| Percent | 78.9 | 8.3 | 1.3 | 9.2 | 2.4 |

The $t$ values corresponding to the four coefficients were 6.4, $-2.03,1.87$, and -4.6. In Equation 1,

```
CP = total number of child pedestrian casualties per zone,
PD = population density (persons per 1,000 m}\mp@subsup{\textrm{m}}{}{2}\mathrm{ ),
SR = ratio of street area per zone to total street area in
        the city (percent), and
CI = number of four-leg (cross) intersections per zone.
```

Model $2\left(n=16, R^{2}=0.75, F=12.37\right)$ :

$$
\begin{align*}
1 /[\ln (\mathrm{CP})]= & 0.55+0.16 \ln (\mathrm{GPS}+1) \\
& +1.43 \ln [(1 / \mathrm{CI})+1] \\
& -0.14 \ln (\mathrm{SR}+1) \tag{2}
\end{align*}
$$

The $t$ values corresponding to the four coefficients were 3.7, $2.9,4.2$, and -1.8 . In Equation 2, GPS $=$ ratio of green areas (parks and playgrounds in thousands of square meters) to the number of elementary schools in each zone (i.e., green area per school).

The two models are statistically significant at a 10 percent level of significance. Both models indicate the effect of fourleg (cross) intersections and street area. Model 2 shows the effect of green areas represented by the area in thousands of square meters divided by the number of elementary schools in each zone. The significance of this factor suggests that in order to achieve a reduction in child accidents, school playgrounds should be open and available to children during the days when school is not in session.

Different models and variables were considered. The models shown were statistically the most significant. Streets in residential areas were not classified according to traffic volumes because data related to their average daily traffic (ADT) were not available. However, population density (which was included in the model) is believed to reflect traffic demand in a specific area.

The first model was used to estimate the expected reduction in child casualties caused by changes in the existing road pattern. A 25 percent decrease in the number of cross intersec-
tions may lead to a 24 percent reduction in child pedestrian accidents. This reduction may be achieved by diagonal or full closure of cross intersections. These methods gave good results when they were implemented in several locations in Europe (8) and Australia (9).

Information related to the destination of the child was not available, that is, whether the child was on the way to school or to the playground. However, such information would not have changed the major findings because few playgrounds exist outside those found at schools.

## CONCLUSION

The results presented provide some evidence to support the hypothesis that a strong relationship exists between city planning and road safety for children. In the city of Zarqa, a significant relationship was found between the number of child pedestrian casualties and population density, road density, green areas per elementary school, and road pattern expressed as the number of four-leg (cross) intersections.
The models presented can be used to predict the expected reduction in child pedestrian casualties from immediate and short-term measures, such as converting four-way intersections to other types of intersections with fewer conflict points.

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## PART 3 <br> Motorist Information System Studies

# Operational Effectiveness of Truck Lane Restrictions 

Fred R. Hanscom


#### Abstract

The operational effectiveness of restricting trucks from designated lanes on multilane roadways is addressed. Three locations with no truck restrictions were treated with signing restricting trucks to certain lanes. The applied field study was of a before-and-after design (with matched control sites). Truck lane restrictions were implemented at two three-lane sites and one two-lane location. Favorable truck compliance effects were evident at all three locations. Before-and-after comparisons indicated significant truck lane use shifts; however, violation rates were higher (i.e., 10.2 percent) at the two-lane site in comparison with the three-lane sites (i.e., 0.9 and 5.7 percent). Higher violation rates at the two-lane site resulted from increased truck densities caused by restricting trucks to a single lane. An emphasis was placed on determining traffic flow effects to nontrucks in the traffic stream. Beneficial effects on three-Iane roadways were realized in terms of reduced congestion and fewer trucks impeding vehicles (at both sites) and shorter following queue lengths (at one site). This finding supports the conclusion that traffic congestion at threelane sites was reduced as the result of the restriction. An adverse effect, observed at the two-lane restriction, was reduced speeds of impeded vehicles following trucks. However, a slight benefit was found in that fewer trucks impeded following vehicles. Allvehicle speed comparisons were examined to determine whether increased differential speeds were likely to occur between the restricted and adjacent lanes. No speed changes were observed to indicate an adverse effect of the truck lane restriction.


The operational effectiveness of restricting trucks from designated lanes on a multilane roadway is addressed. Three study sites with no truck restrictions were treated with signs similar to those shown in Figure 1. The study procedure used was the before-and-after method (with matched control sites without signs). The primary measure of effectiveness (MOE) for lane restriction was voluntary truck compliance with the restriction (data were not collected in the presence of enforcement activity). Additional MOEs addressed relevant traffic flow conditions affected by the restrictions. These conditions were (a) traffic congestion as determined from speeds and platooning behaviors for vehicles following trucks, and (b) differences in speeds between the restricted and adjacent traffic lanes for all vehicles.

## STUDY LOCATIONS

The truck lane restrictions were implemented at three locations designated by participating states. Two fringe-area urban sites near Chicago were restricted by extending previously existing lane restrictions. The purpose of the Chicago area

[^15]lane restrictions was to improve traffic flow and operational safety. In addition, one rural, two-lane Interstate in Wisconsin was treated with a right-lane restriction because of deteriorated pavement. The Chicago restrictions prohibited trucks from using the left-hand lane of a three-lane facility, whereas the Wisconsin site restricted trucks from the right-hand lane of a two-lane roadway.
Specific site information follows.

## Wisconsin

I-90/I-94 eastbound, near Lake Delton; average daily traffic 4,478, 13.4 percent trucks; restriction location Milepost 93105; and control location 3 mi east of restriction.

## Illinois

I-55 eastbound, Du Page County; average daily traffic 23,500 eastbound, 21 percent trucks; restriction location west of County Line Road; and control location 2 mi west of restriction.
I-290 eastbound, Addison; average daily traffic 78,500, 13 percent trucks; restriction location west of Wolf Road (Milepost 747.2); and control location 2 mi west of restriction.

Similar geometric alignment conditions existed at all study and control sites. These sites consisted of tangent sections with minimum sight distances of $1 / 2 \mathrm{mi}$. Sites (e.g., truck stops and industrial areas) were selected on the basis of low truck exit entry activity and were located at sufficient distance (i.e., 1-mi minimum) from ramps to effects of exit and entry activity on lane distribution.

## EXPERIMENTAL APPROACH

The approach used was a before-and-after study with control sites. Identical behavioral observations were made at geometrically matched nonrestricted (control) sections on the same highway as the restricted (test) sites. Locations of sites within designated pairs, each containing an upstream control section, virtually ensured measurement of the same truck sample at the test and control locations.
Data collection was conducted on weekdays and was strictly controlled for a time-of-day match between before and after conditions. In order to minimize seasonal, traffic volume, and traffic mix effects, an attempt was made to conduct before and after observations exactly 52 weeks apart. However, this feature was not possible for one site (I-290 in Chicago) because of state agency timing for implementing the new restriction. Yet, concurrent observations at the matched control site did ensure the integrity of the applied experimental procedures.

## TRUCKS USE 2 RIGHT LANES

FIGURE 1 Applied truck restriction signing in the Chicago area.

## MOEs

Applied operational MOEs for evaluating the three cases of truck lane restrictions were

- Truck lane occupancy,
- Delay to following vehicles,
* Proportion of trucks impeding followers, and
- Adjacent lane speed differential.

The following is a brief discussion of field measurement procedures and significance of these MOEs.

## Truck Lane Occupancy

Because the regulatory intent of the lane restriction was to preclude trucks from designated lanes, compliance measures consisted of truck counts by lane before and after the restriction was placed in effect. Manual counts were conducted at restricted and nearby control sites. Before-and-after observations were matched by time-of-day and day-of-week.

## Delay to Following Vehicles

Operational effects of restricting trucks to certain lanes involve highway capacity and congestion. Thus, the target was not truck speeds per se, but rather speeds and queuing (i.e., platooning) characteristics of vehicles in the stream that were affected by trucks. The primary operational concern associ-
ated with restricting trucks to specific lanes is whether greater delays and longer queues occur in nonrestricted lanes for vehicles following trucks.

The data collection approach used is shown in Figure 2. Results from previous research (1) have determined specific vehicle following distances associated with free-flow speeds. Absence of a free-flow condition results in the queuing of vehicles because the speeds of following vehicles are impeded by leading vehicles. Therefore, the lane restriction sites were instrumented to support visual observation to determine whether vehicles following trucks were queued or free-flow conditions existed. Unobtrusive roadside markers were applied to measure the number of queued vehicles following target trucks. When a second truck was queued behind a lead truck, it merely counted as a queued vehicle. Referring to Figure 2, counts were made of following vehicles with headways less than $P$, the following distance associated with platooning in the absence of free-flow conditions. A following vehicle with a headway equal to or greater than $P$ signified the end of the queue.

Speeds for the queues were determined using a modified version of the radar-platooning technique (2). Because radar could be detected by truck operators and consequently bias results, manual speed timing was applied to measure speeds for vehicles following trucks (i.e., given that the observed headway was less than $P$ ). The speed timing procedure applied had been previously validated and produced a sample accuracy of 0.1 mph (3). Application of these procedures for measuring platoon speed involved clocking speeds of the lead vehicle and multiplying by the number of queued vehicles to determine a weighted mean speed for the overall sample.

## Proportion of Trucks Impeding Followers

An applied measure of congestion is the before-and-after proportions of trucks that impeded following vehicles. As can be seen from Figure '2, when no vehicle is within following distance behind a target truck, this truck is not impeding a follower. This measure provided a basis for examining the operational effect of less congestion attributed to greater ease of passing trucks in a lane-restricted flow situation. The pro-


FIGURE 2 Designated vehicles for field data collection.
portion of trucks associated with zero queue lengths represents those trucks not impeding following vehicles.

## Differential Speeds Between Lanes

Large differential speeds between restricted and adjoining traffic lanes are a safety concern because of the increased accident potential for vehicles that change lanes. Therefore, a major operational issue in the assessment of restricted truck lanes is whether an increased speed difference results between restricted and adjacent, nonrestricted lanes. This safety concern evolves from two potential operational effects that cause increased differential speeds between restricted and adjacent lanes: (a) absence of trucks may produce an overall speed increase in the restricted lane, and (b) an increase in number of trucks may result in a decrease in speed in the adjacent lane.

Differential speeds between lanes represent a safety hazard (or accident potential) under conditions of sufficient traffic density. The following two conditions can be used to measure any potential effect: (a) presence of trucks influencing overall stream speeds, and (b) presence of improper lane changing. The data collection procedure applied to determine the MOE for speed differences was all-vehicle speed sampling in the restricted and adjacent lanes.

## RESULTS

Data for primary MOEs (truck lane occupancy, trucks impeding followers, and following vehicle delay) were gathered at all three truck lane restriction locations. The MOE for differential speed between lanes required sufficient left-lane traffic volume that removal of trucks could be expected to affect allvehicle speeds. For this phenomenon to occur, a sufficiently high traffic volume was necessary to permit vehicle interactions between trucks and other vehicles. This prerequisite traffic condition existed at only one of the test sites. Therefore, results based on lane occupancy, trucks impeding followers, and following vehicle delay are discussed separately from results based on between-lane speed differentials.

## Based on Lane Occupancy, Trucks Impeding Followers, and Follower Delay MOEs

The two Chicago area study conditions were left-lane restrictions on three-lane roadways, whereas the Wisconsin site comprised a right-lane restriction on a two-lane roadway.

## 1-290, Chicago Area

Before-and-after comparisons for the following traffic flow parameters were observed for test and control site pairs.

- Average Hourly Traffic Volume. Total vehicle volumes were estimated on the basis of 5 -min periods of continuous counting during each hour of traffic observation.
- Truck Distributions. Observed truck counts (and percentages) by lane, indicating which lane was restricted in the after condition.
- Following Vehicle Speeds. As previously indicated from Figure 2, speeds of vehicles queued behind trucks were obtained by means of the platoon-weighting technique.
- Following Queues. Average number of queued vehicles behind observed trucks is given as a measure of before-andafter traffic congestion.
- Truck Proportion Impeding Followers. Proportion of trucks in the stream characterized by platooned vehicles queued behind them.

Before-versus-after truck occupancy by lane was seasonally affected by an increase in traffic volume. Observed traffic volume increases were approximately 23 and 13 percent at the restricted and control sites, respectively. These differences contributed to an increase in truck occupancy of the left lane at the control site from 3.8 to 5.4 percent; however, a concurrent slight reduction of left-lane occupancy, from 6.7 to 5.8 percent, was observed at the restricted site. This relative difference in directionality between the test and control locations was caused by the lane restriction.

Although neither of the before-and-after truck lane occupancy shifts at either the test or control site was statistically significant taken separately (from application of $z$-test of proportions), combined effects considering changes at both locations (from application of chi-squared contingency tests) demonstrated statistical significance at the 99 percent confidence level.

An application of the chi-squared statistic did reveal differences between the test and control sites during the before condition. Application of the odds ratio directly compared the probability of left-lane traffic presence between the test and control sites (i.e., an odds ratio of 1.775 , with a standard error of 0.2962 , indicated a factor of 1.775 greater probability of a truck's being in the left lane at the test site). However, the same statistical procedure applied to data collected in the after condition indicated a nearly equal likelihood of left-lane truck occupancy at either the test or control site. The combined effect determined from these two observations is that

TABLE 1 BEFORE-AND-AFTER CHANGES FOR CHICAGO AREA I-290 SITE

| Site | Average Hourly Truck Volume (\%) | Right- <br> Lane <br> Truck <br> Distribution <br> (\%) | FollowingVehicle Speed (mph) | Following Queues (veh) | Truck <br> Fraction <br> Impeding <br> Followers <br> (\%) |
| :---: | :---: | :---: | :---: | :---: | :---: |
| Control | +13.0 | +1.6 | -0.6 | $+0.43$ | + 15.2 |
| Test | +23.0 | $-0.9{ }^{\text {a }}$ | -0.7 | $+0.14^{a}$ | +3.2 ${ }^{\text {a }}$ |

[^16]the lane restriction applied at the test site was effective at reducing left-lane truck usage.

Average flow delay to vehicles impeded by trucks was recorded both at the test and control locations. Small average speed reductions between before and after conditions ( 0.6 mph at the control site and 0.7 mph at the restricted site) were statistically significant at the 0.01 probability level. This change is interpreted as a general slowing because of the traffic volume increase during the after condition.

In order to illustrate the effect on vehicle queuing behind trucks, Figure 3 shows before-and-after percentage distributions for observed queue lengths both at the control and test sites. Of particular interest is the proportion of trucks that do not impede following vehicles, i.e., of trucks characterized by a zero length of following queue. At the control site, a significantly smaller proportion of zero queue length, 37.2 versus 45.4 percent, signifying a more congested condition, was observed during the after condition; whereas no significant differences in queuing were observed at the test site. Thus, a benefit of the lane restriction was realized in terms of no corresponding increase in queuing at the test site.

The zero-queue proportional differences were statistically determined by application of the $z$-test. Chi-squared tests applied to overall queuing distributions confirmed generally longer queues in the after condition at the control site. This demonstrated operational effect shows that restricting trucks
to the right-hand two lanes improved the passing ability of following vehicles. Although more congestion (e.g., longer mean queue lengths) was observed both at the test and control sites because of higher traffic volumes in the after condition, the relative increase (i.e., greater congestion at the control site) provides additional evidence of the effectiveness of truck lane restrictions.

Table 2 presents results of the data analysis previously discussed for the I-290 site pair. The operational effect of the lane restriction was a decrease in left-lane truck occupancy, shorter queues following trucks, and fewer trucks impeding following vehicles. A greater relative following queue length reduction was observed at the test site (by comparison with the control) despite a larger increase in traffic volume. Thus, the demonstrated benefit of the truck lane restriction was an overall traffic congestion reduction. No sustained effect on speeds of vehicles following trucks was attributable to the lane restriction.

## I-55, Chicago Area

Observations were made at a second Chicago area truck lane restricted site. I-55 is characterized by a lower traffic volume condition in which no left-lane trucks were observed at either the test or control sites during the before condition. As pre-


FIGURE 3 Before-and-after queue length distributions for I-290, Chicago site pair.

TABLE 2 BEFORE-AND-AFTER CHANGES FOR CHICAGO AREA I-55 SITE

|  | Average    <br>  Hourly Right- Lane <br>  Truck Truck Following- <br>  Volume Distribution Vehicle <br> Speed <br> $(\mathrm{mph})$ <br> Site $(\%)$ $(\%)$ Following | Queues <br> (veh) | Frackion <br> Impeding <br> Followers <br> $(\%)$ |  |  |
| :--- | :--- | :--- | :--- | :--- | :--- |
| Control | +21.0 | +2.1 | -0.6 | -0.01 | -6.2 |
| Test | +7.0 | $+0.9^{a}$ | -2.7 | -0.07 | $-11.8^{a}$ |

[^17]viously noted for the I-290 site, an increase in traffic volume affected truck presence in the left (restricted) lanes during the after condition. However, application of the chi-squared statistic indicated that the probability of a truck's being in the left lane during the after condition at the restricted site was only 0.43 that of being in the left lane at the control site. This analysis demonstrates a favorable effect of decreased truck usage of the restricted lane.
Also, as in the case of I-290 site pairs, similar flow differences were observed between before and after conditions both at the test and control locations. Both for the test and control sites, slight but statistically significant speed increases were noted for vehicles queued behind trucks. This effect may be expected to accompany less stable flow associated with higher traffic volumes. However, similar differences occurring at both the test and control sites substantiate that no detrimental flow effects were attributable to implementation of the lane restrictions.
Speed changes of vehicles following trucks both at the test and control sites were examined for a possible effect of the lane restriction. However, observed differences could be attributed to the before-and-after traffic volume increase rather than truck lane restriction effects.
Before-and-after queuing differences for vehicles impeded by trucks can be seen in Figure 4, which shows plots of before-and-after frequency distributions for the test and control sites. The before-and-after proportion of trucks not impeding other vehicles increased from 44.9 to 51.4 percent at the test site,
whereas no significant change was observed at the control site. Although following-vehicle queue lengths were less varied in the after condition, the observed tendency to relatively shorter queues (by comparison with the control site) could not be statistically sustained as a benefit of the restriction. However, the overall improved flow condition, characteristic of a less congested situation, is attributed to implementation of the truck lane restriction.
Data analysis results are presented in Table 3. Lane restriction effects observed at I-55 were consistent with I-290 find-ings-significantly reduced truck usage of the left lane was accompanied by fewer trucks impeding following vehicles. Thus, the resulting operational effect was reduced overall traffic congestion attributable to implementation of the lane restriction.

## I-90/I-94, Wisconsin

Traffic conditions were observed at a third site pair on I-90/ I-94 near Madison, Wisconsin. This restriction test differed from that in Chicago area sites in that the site consisted of a two-lane geometry with a right-lane restriction. Thus, the before condition was characterized by 87.4 percent of the observed trucks in the restricted lane. An observed truck percentage reduction to 10.2 percent in the after condition was significant, and no concurrent reduction occurred at the control site.


FIGURE 4 Before-and-after queue length distributions for I-55, Chicago site pair.
TABLE 3 BEFORE-AND-AFTER CHANGES FOR I-90/I-94 WISCONSIN SITE

|  |  |  |  |  |  |
| :---: | :---: | :---: | :---: | :---: | :---: |
|  | Average Hourly | Right- <br> Lane | Following- |  | Truck <br> Fraction |
|  | Truck | Truck | Vehicle | Following | Impeding |
|  | Volume (\%) | Distribution | Speed (mph) | Oueues (veh) | Followers (\%) |
| Control | -4.2 | -4.3 | +1.2 | -0.11 | -23.4 |
| Test | -6.7 | $-77.2^{a}$ | $-1.4{ }^{\text {a }}$ | -0.09 | -25.1 |

[^18]Violations consisting of trucks traveling in the restricted lane of the right-lane restriction ( 10.2 percent) were more frequent than those observed at either Chicago left-lane site (i.e., 5.7 and 0.9 percent, respectively). However, this result was caused by the restricting of a two-lane as opposed to a three-lane roadway geometry. Because the study sites were characterized by substantial truck volumes, less compliance could be expected at a two-lane site because of the necessary crowding of trucks.

A slight, but statistically significant, overall speed decrease in the after condition was observed for impeded vehicles following trucks at the test site. This effect was caused by the denser truck traffic that was restricted to a single lane. This speed decrease is especially noteworthy in view of a concurrent speed increase observed at the control site. Reversal of following-vehicle speeds (i.e., faster at the control and slower at the restriction) was not observed at either Chicago site.

Lane-specific speed analyses for following vehicles indicated a major expected effect of the right-lane truck restric-tion-significant slowing in the left lane was associated with the increased truck presence. However, trucks remaining in the right-hand lane at the test site that violated the restriction also exhibited reduced speeds in the after condition. This effect was associated with the lane restriction. The opposite effect (i.e., increased right-lane speeds) was observed at the control. Two likely explanations of the speed reduction at the test site were (a) restricted passing opportunities, and (b) driver uncertainty resulting from high left-lane truck presence that violated driver expectation for the lane carrying slowermoving vehicles.

Overall following-vehicle speed reductions were seen as an operational effect of restricting trucks from the right-hand lane. Increased left-lane truck congestion restricted passing opportunity and created uncertainty for those vehicles queued behind remaining right-lane trucks. These effects are viewed as negative impacts of the right-lane truck restriction.

Figure 5 shows before-and-after queue length distributions at the test and control sites. Significantly larger percentages of trucks were observed not to impede following vehicles during the after condition both at the test (79.7 versus 72.1 percent, respectively) and control ( 79.0 versus 72.6 percent, respectively) sites. Although this result is likely because of lower truck volumes, a slightly stronger statistical relationship (from application of the omega-squared statistic) was noted at the test site. Thus, the finding of less queuing behind trucks, noted at the previously discussed sites, also applies to the Wisconsin site pair.

Results of the Wisconsin site data analysis are presented in Table 4. Although implementation of a right-lane truck restriction was effective in eliciting a significant shift in truck lane presence, certain adverse effects (e.g., greater slowing of following traffic) likely resulted from restricted passing opportunity hecause of the crowding of trucks into the left lane. Therefore, the congestion-reducing benefit previously observed with truck restrictions imposed on a three-lane roadway was less evident in this two-lane situation. Finally, an inherent concern with a two-lane restriction is increased violation rates and congestion associated with higher truck volumes.

## Based on the MOE of Differential Speed Between Lanes

Before-versus-after speeds were investigated to determine whether restricting trucks from one lane increased the speed differential between that and the adjacent lane.

Of the three truck lane restrictions, two were characterized by sufficiently low traffic volumes and densities that did not exert an influence on overall traffic speeds. Therefore, the differential speed study was limited to a one-lane restriction site pair (on I-290, near Chicago).


FIGURE 5 Before-and-after queue length distributions fo: I-90/I-94, Wisconsin site pair.

TABLE 4 ALL-VEHICLE SPEEDS OBSERVED ON RESTRICTED AND NONRESTRICTED EXPRESSWAY SECTIONS NEAR CHICAGO

|  | Restricted Lane |  |  |  | Adjacent Lane |  |  |  |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: |
|  | $N$ | Mean <br> (mph) | Standard Deviation (mpil) | 99 Percent Confidence Interval (mph) | $N$ | Mean <br> (mph) | Standard Deviation (mph) | 99 Percent Confidence Interval (mph) |
| Test |  |  |  |  |  |  |  |  |
| Before | 434 | 62.2 | 4.66 | 0.52 | 567 | 59.3 | 4.63 | 0.45 |
| After | 442 | $60.6^{\text {a }}$ | 4.69 | 0.51 | 671 | $58.4{ }^{\text {a }}$ | 4.18 | 0.38 |
| Control |  |  |  |  |  |  |  |  |
| Before | 511 | 64.6 | 4.32 | 0.44 | 587 | 59.7 | 3.67 | 0.35 |
| After | 461 | $62.0^{a}$ | 4.03 | 0.44 | 643 | $58.7{ }^{\text {a }}$ | 4.04 | 0.37 |

${ }^{a}$ Significant change between before and after condition at the 99 percent confidence level.

The study procedure involved sampling all-vehicle speeds (in which any vehicle, regardless of type, had an equal probability for inclusion into the sample) both in the restricted lane and adjacent lane at various times throughout two separate days both in the before and after conditions. Observations were limited to stable and free-flow conditions (at Level of Service C or better). During unstable or forced-flow conditions, the effect of differential speed would have been masked, thereby rendering impossible measurement of any effect of the restriction. Speeds were manually timed by electronic stopwatch. A randomization procedure was applied to eliminate coder bias in selecting vehicles for speed measurement. This technique had been previously validated and found to produce sample results within 0.1 mph of all-vehicle population speeds (3).
Summary speed observations are presented in Table 5. Sufficient sample sizes were gathered at all locations to support mean speed determination with an accuracy of 0.5 mph at the 99 percent confidence level. Controlled day-of-week and time-of-day observations ensured uniformity of flow conditions between before and after periods. Thus, any seasonal effect was controlled by application of the test and control site study design.
Table 5 indicates that mean speeds were significantly lower both at the test and control sites in the after period. Weighted average speeds (considering relative volumes of the restricted and adjacent lanes) were approximately 1.9 mph lower at the control site and 1.3 mph lower at the test site. No differences in overall speed variation were observed. The apparent volume-related effect of lower mean speeds exceeded any observable effect of the truck restriction. Observed mean differential speeds between restricted and adjacent lanes for the before and after conditions are as follows:

| Scenario | Test $(\mathrm{mph})$ | Control $(m p h)$ |
| :--- | :--- | :--- |
| Before | 2.9 | 4.9 |
| After | 2.2 | 3.3 |

An adverse effect of truck lane restrictions was an overall speed increase in the left lane accompanied by a possible speed reduction in the adjacent lane. This occurrence would produce higher differential between-lane speeds and a possibly greater accident hazard. However, as noted previously, a general speed reduction was observed in the after condition; furthermore, speed differences across lanes did not increase.

Therefore, no adverse speed effect could be attributed to the shift in truck occupancy,

Mean lane-specific changes between the before and after conditions are as follows:

| Lane | Test (mph) | Control (mph) |
| :--- | :--- | :--- |
| Restricted | -1.6 | -2.6 |
| Adjacent | -0.8 | -1.0 |

Although significant before-and-after speed decreases were greater in the left lane, the change was less pronounced at the test site. This finding indicates no adverse effect of the lane restriction in mean speed change because safety (as associated with smoother flow) is enhanced by the less severe before-and-after speed difference.

## DISCUSSION OF RESULTS

Truck lane restrictions were implemented at two three-lane sites and one two-lane location. The left lane was restricted at the three-lane sites, whereas the right lane was restricted at the two-lane location. Timing and locations of observed restrictions depended on state highway agency decisions and could not be controlled.
Favorable truck compliance was evident at all three restriction locations. Before-and-after comparisons, undertaken at matched test and control site pairs, indicated significant truck lane changes in compliance with all three restrictions. However, violation rates were 10.2 percent higher at the two-lane site versus 0.9 and 5.7 percent higher at the three-lane sites. This lower level of compliance likely resulted from high truck concentrations, because of the restricting of trucks to a single lane. No indication was found that differential compliance behavior was associated merely with left- versus right-lane restrictions.

The emphasis of the procedure was to determine flow effects to nontruck vehicles in the traffic stream. The primary MOE was delay to impeded vehicles. Beneficial traffic flow effects resulted from lane restrictions applied to three-lane roadways. Under this geometric condition, reduced traffic congestion was realized in terms of fewer trucks' impeding vehicles at both sites and shorter following queue lengths at one of the two sites. This finding is based on relative effects between matched test and control site pairs. MOEs related to traffic
flow (such as impeded queue lengths) benefitted despite increased traffic volumes in the after condition that would ordinarily suggest degraded flow conditions. The observed improvement in flow was further evidence of the benefit of the truck lane restriction. The significance of this finding is that implementation of truck lane restrictions at three-lane sites did achieve the generally intended goal of reducing the overall level of congestion.

An adverse flow effect observed at the site with the twolane, right-hand-lane restriction was reduced speeds of impeded vehicles following trucks. Operational causes of this finding were crowding of trucks into the left lane in combination with limited passing opportunity for remaining right-lane truck followers. Concurrent control site observations confirmed that this effect was because of implementation of the restriction. A weak statistical finding indicated a slight benefit in that fewer trucks impeded following vehicles at the two-lane site.

All-vehicle speed comparisons were examined at one location to determine whether increased differential speeds were likely to occur between the restricted and adjacent lanes. This investigation was prompted by a concern that overall speeds would increase in the restricted lane and decrease in the adjoining lane. No speed changes were observed to indicate an adverse effect of implementing the truck lane restriction.

## CONCLUSION

Beneficial traffic flow effects (e.g., reduced congestion) associated with left-lane truck restrictions on three-lane roadways support their continued use. However, findings including high violation rates and slowing of impeded vehicles associated
with the two-lane site restriction raise safety issues that warrant an accident study or other further investigation.

## ACKNOWLEDGMENTS

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[^19]
# Motorist Comprehension of Signing Applied in Urban Arterial Work Zones 

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#### Abstract

Motorists encounter numerous problems that are not currently addressed in traffic control manuals for urban work zones. The motorists' understanding and properly attending to signing in these work zone areas is critical to ensure safe operations. A detailed survey about Farm-to-Market (FM) 1960 in Houston, Texas, investigated motorists' comprehension of construction signing. FM 1960 is a four-lane, undivided major arterial with a continuous left-turn lane. The continuous left-turn lane was excluded during construction because of a restricted right-of-way. The survey was designed to meet three objectives: (a) to ascertain knowledge about work zone signing, (b) to determine problem areas of the FM 1960 signing, and (c) to elicit information from motorists concerning overall problems with the FM 1960 project. Personal interviews were conducted with 205 respondents from the FM 1960 areas. These participants were asked to respond to questions about work zone signing and other forms of traffic control devices. The response percentages revealed that motorists have some difficulty interpreting both word and symbol messages on signs. This lack of comprehension intensifies the existing problems in work zone areas and indicates a need for further research to improve urban arterial operations.


As a result of continued increase in congestion on many freeways in major Texas cities, the arterial street systems next to these freeways are being forced to carry a larger load of the traffic burden. In September 1987, the Texas State Department of Highways and Public Transportation implemented a $\$ 100$ million program, the Principal Arterial Street System (PASS), to upgrade many urban arterials to provide additional capacity and improve traffic flow. However, construction traffic control on arterials in highly developed urban areas presents many problems not currently addressed by signing in the Manual of Uniform Traffic Control Devices (1). These problems include increased driver workload associated with construction sign requirements, constricted rights-of-way, moderate speed and volume combinations, compressed spacing between signalized intersections, heavy protected and unprotected turning movements, and excessive provisions for access to adjacent property (driveways).

Vehicle speeds are typically lower in urban areas than in rural areas. A driver may have more time available to comprehend the traffic environment. However, the high density of signing and congestion on a typical urban arterial will usually offset the advantage of lower operating speeds.

Other literature $(2,3)$ indicates a large discepancy between the amount of applicable signing for freeway and rural highway work zones and that recommended for use in urban arterial work zones. One study (4), which evaluated the implementation of traffic control plans, specifically cited motorist

[^20]confusion with signing in urban arterial work zones. Motorist misunderstanding of signing, in general, was documented by the Texas Transportation Institute (5).

Because of the special problems associated with urban arterial work zones, motorists traveling these roadways must comprehend and properly attend to the signing used for traffic control. This understanding is critical to ensure that communication in these areas promotes good operations and safety. Therefore, the level of motorist comprehension of signing within an urban arterial work zone was assessed.

## INTRODUCTION

FM 1960 is included in the Farm-to-Market (FM) road program, which was approved by the Texas legislature as a classification of highways in the state of Texas. Reconstruction of FM 1960 in Houston, Texas, which began in January 1988, proceeded from Interstate 45 to State Highway 249, a 7-mi segment of roadway (Figure 1). FM 1960 consists of four undivided lanes with 100 ft of right-of-way. The reconstructed facility will include six lanes with a continuous left-turn lane. The primary land use along this major arterial is retail and residential. The traffic volume before construction was approximately 45,000 veh/day. Approximately 360 driveways and 27 signalized intersections are within the project limits.

All aspects of the implemented traffic control plan met or exceeded the requirements of the Texas Manual on Uniform Traffic Control Devices (6). However, complaints and comments from FM 1960 motorists and adjacent retailers suggested that the volume and speed of traffic, as well as the number of turning vehicles, posed significant problems warranting study. An origin-destination and opinion survey (7) confirmed motorist confusion and poor understanding of signing through the reconstruction area. On the basis of this information, a detailed survey was initiated in 1989 to investigate motorist comprehension of construction signing within this corridor.

## STUDY DESIGN AND METHODOLOGY

A survey was designed to meet the following objectives:

- Ascertain knowledge about work zone signing in generai,
- Determine confusing or problematic areas of the FM 1960 signing in particular, and
- Elicit information from motorists concerning problems with the FM 1960 project that may not be related to understanding traffic control devices.


FIGURE 1 Construction project map.

More specifically, three questions were posed: (a) Are motorists having difficulties with the construction area because of confusion or the number of signs and traffic control devices or both? (b) Are motorists having trouble finding destinations in the construction area because of problems with signing? and (c) Are primary FM 1960 users primarily concerned about traffic control and signing, or are other factors more important?

Personal interviews were conducted with 205 respondents in February 1989 at two locations, Willowbrook Mall and the Grant Road Texas Department of Public Safety (DPS) Licensing Office. Response was strictly voluntary in the mallpotential respondents were not approached randomly. However, respondents at the licensing office were asked to participate in the study. The result was that 115 respondents were interviewed at the DPS licensing office and 90 respondents were interviewed at Willowbrook Mall.

Survey participants were first asked to respond to questions about work zone signs and other forms of traffic control devices presented in a booklet of photographs. This set of questions was followed by a series of photographs or signs and scenes from the FM 1960 reconstruction project with corresponding
questions. The third segment of the interview was a discussion with the respondent about his or her opinion on various aspects of the reconstruction project. A brief set of biographical questions concluded the interview. The interview took an average of 10 min . The work zone signing questions are shown in Figure 2.

## RESULTS

As anticipated, the survey revealed that drivers have some difficulty interpreting hoth word and symbol messages on signs. Response percentages for each of the sign questions are presented and discussed by category.

## Road Construction 500 FT

Two-thirds ( 66.0 percent) of the survey respondents correctly interpreted the sign shown in Figure 3 to indicate an advance warning of construction 500 ft ahead. However, one-fourth

What does this sign mean?

1. A. Road construction ahead
B. Flagger ahead
C. Guard for school crossing ahead
D. Not Sure
2. A. Leave room for traffic crossing at intersection
3. If your car stalls, move it out the of intersection
C. Move through the intersection quickly
D. Not Sure
4. A. Drive in the center, the lane is not marked
B. Drive in the right lane only
C. Be alert for cars stopping to turn left
D. Not Sure
5. A. Left turn lane marker
B. Left lane ends
C. Median narrows
D. Not Sure
6. What do the orange and white striped signs mean?
A. Do not turn between these signs
B. Pay special attention to signs on these posts
C. Drive to the right of these signs
D. Not Sure
7. What do the orange and white posts on the right tell you?
A. Hazardous area to the right, drive to the left of posts
B. Shows the right edge of the pavement
C. Park between these posts
D. Not Sure
8. Are you permitted to turn left in front of the barrel with the crossover sign?
yes
not sure $\quad$ ___ no
other
9. Do you think signs like the Auto tint sign should be allowed in the construction area?
$\qquad$ yes
not sure $\qquad$ no
10. Why are these signs different colors?
11. What is your opinion of these red signs?
12. A. Median narrows
B. Right lane ends
C. Right turn lane marker
D. Not Sure
13. A. Divided road ahead
B. Obstacles in the road ahead
C. Merging traffic ahead
D. Not Sure
14. A. Low shoulder
B. Uneven pavement
C. Bumpy road
D. Not Sure
15. A. Drive in the outside lane only
B. You cannot go straight at the next light
C. A Lane for Left Turns is not provided
D. Not Sure
16. What does the green sign mean?
A. Crossover here
B. Crossover at the next signal
C. Emergency vehicle cross here
D. Not Sure
17. What does the second yellow sign mean?
A. Obstacles in the road ahead
B. Merging traffic ahead
C. Divided road ahead
D. Not Sure
18. Are you permitted to turn left behind the barrel with the crossover sign?
$\qquad$
19. Are you permitted to turn right at this intersection?
___ yot sure not other
20. You are driving the pickup, what should you do at this intersection?

FIGURE 2 Sign questionnaire summary.
(25.2 percent) of the respondents interpreted the sign as the beginning marker for a construction area that would continue for 500 ft .
Respondents viewed the same sign in a photographed segment of FM 1960. In the context of the construction area, the percentage of correct interpretations did not increase. In response to the sign shown in Figure 4, 33 percent of those surveyed said that the next 500 ft of roadway are under construction, and 58.3 percent said construction would be encountered 500 ft ahead.

## Advance Flagger Symbol Sign

The Flagger Ahead symbol sign was interpreted correctly more often in the construction context, presented by photograph, than out of context. Figures 5 and 6 show the signs as presented to the survey respondents. The symbol sign of context was correctly interpreted by 77.5 percent of the respondents. In context, correct interpretation increased to 85.1 percent. Most of those who misinterpreted this sign said that it indicated road construction ahead.


FIGURE 3 Sign indicating advance warning of construction.


FIGURE 4 Advance warning sign shown on FM 1960.


FIGURE 5 Flagger Ahead symbol sign.


FIGURE 6 Flagger Ahead sign in construction context.

## Low-Shoulder Symbol Sign

The correct interpretation rate of the low-shoulder symbol sign (shown in Figure 7) was very poor. Most drivers (84 percent) thought that this sign indicated uneven pavement, rather than low shoulder.

## Lane Ends Symbol Sign

Median Narrows was selected by 15.7 percent of the respondents as the meaning of the symbol sign shown in Figure 8. Right Lane Ends, the correct meaning, was chosen by 78.4 percent of the respondents. In response to the photograph in Figure 9, 9.9 percent thought the sign was a left-turn lane marker. The correct response was given by 79.2 percent of the respondents.

## Response to Sign Messages

Respondents were asked to describe the appropriate driving response to several regulatory and informational signs posted


FIGURE 7 Low-Shoulder symbol sign.


FIGURE 8 Right Lane Ends symbol sign.


FIGURE 9 Right Lane Ends sign shown in context.
in the construction area. The results showed that, for some signs, a clear and single message was not delivered.

No Center Lane and No Center Turn Lane signs (Figures 10 and 11) are used throughout the FM 1960 construction area. These signs were confusing to many respondents. Fortysix percent believed they should drive only in the right lane in response to the No Center Lane sign, and 15 percent thought No Center Turn Lane meant that they should not turn from the center lane. Only 46.1 percent checked the appropriate response - "be alert for cars stopping to turn left."

Green Crossover signs, when posted on a free-standing barrel as shown in Figure 12, do not clearly convey to the motorist where to cross over. Survey participants were asked whether crossing over is permitted before and after the Crossover sign. In response to the situation shown in Figure 12, 55.2 percent said that crossing over was permitted in front of the barrel; 38.4 percent said it was not. Eight percent of the respondents said they were not sure if they would be permitted to turn behind the barrel with the Crossover sign, 43 percent said that they could turn behind it, and 49 percent said that they could not turn behind it. A ribbon barrier attached in front of the barrel as shown in Figure 13 simplified the response. In this case, 82.3 percent of those surveyed said that turning left in front of the Crossover sign was not permitted, and 80.2 percent said that turning left behind the sign was permitted.


FIGURE 10 No Center Lane sign.


FIGURE 11 No Center Turn Lane sign.


FIGURE 12 Crossover sign.


FIGURE 13 Crossover sign with ribbon barrier.

Four response choices were provided for the Do Not Block Intersection sign out of context. Response frequencies for each answer were as follows:

1. Leave room for traffic crossing at intersection - 73.5 percent,
2. If your car stalls, move it out of the intersection- 9.8 percent,
3. Move through the intersection quickly- 15.7 percent, and
4. Not sure- 1.0 percent.

Figure 14 shows the sign as presented in context. Respondents were asked what they thought they should do if they were the driver of the pickup truck. In this situation, 99.0 percent described in their own words an appropriate driving response. Again, only 1 percent said they did not know what to do in response to this sign.

## Color Cues

Results from this survey support other research findings that color coding to distinguish construction signs from other types of signs is not well recognized by the motoring public. When shown two two-way traffic symbol signs, one yellow and one orange, and asked the meaning of the two different colors, over 40 percent said that they did not know. Several respondents remarked that they did not believe they had ever seen orange signs. Only 44 percent knew that orange is the color designated for construction signs.

For some segments of the construction area, orange-andwhite hazard markers were used at the pavement edge, as shown in Figure 15. Although 70 percent of the respondents thought these markers indicated a hazardous area to the right, 26 percent thought they marked the right edge of the pavement. In contrast, solid white markers (Figure 16) were used in the construction area as pavement edge markers. The percentage of drivers who recognized these as pavement markers was 58.3, whereas 35.9 percent interpreted them as hazard markers.


FIGURE 14 Do Not Block Intersection sign in context.


FIGURE 15 Orange-and-white hazard markers used in construction area.


FIGURE 16 Solid white markers used in construction area to mark edge of pavement.

## Turning and Finding Destination

The second major question addressed in the survey was whether motorists were having difficulty locating or getting to their destinations because of the construction or signing in the construction area. After answering the questions with fixed choice responses, survey respondents were interviewed for their opinions on a variety of FM 1960 construction problems (Figure 17). Specifically, they were asked, "Do you have trouble finding certain places you want to go because of the construction?" Half ( 49.5 percent) of the respondents said yes to this question, and half ( 50.5 percent) said no.

Subsequently, drivers were asked, "Are there too many, too few, or the right amount of signs that give directions to places alongside the construction area?" The response given most often ( 48.8 percent) was that the right amount of directional signs is being used. However, 28.9 percent said that too few and 18.4 percent said that too many signs were present.

To optimize the visibility of their businesses and entrances to their businesses, some retail owners adjacent to FM 1960 posted directional signing. In most cases, these signs pictured the business name, logo, and an arrow (as shown in Figure

1. How much did construction delay you in getting
to the mall/driver license station today?
(3 most frequent response percentages)
10 min. -- 20.7\%
15 min. -- 22.2\%
20 min. -- 18.5\%
3a. Do you drive on FM 1960 to work or other places during rush hour?
yes -- 45.2\% no -- 54.8\%

3c. Would you say that amount of delay is unreasonable?
yes -- 46.1\% no -- 52.8\% other 1.1\%
5a. Do you think the benefits of widening this road will be worth the inconvenience now?
yes -- 90.7\% no -- 7.3\% other -- 2.0\%
6. Do you have trouble finding specific places you want to go because of the construction?
yes -- 49.5\% no -- 50.5\% other
8. Are there too many signs, too few signs, or the right amount of signs that give information about how to drive through the construction area?

```
too many -- 18.4% too few -- 28.9%
right amount -- 48.8% Comments -- 3.9%
```

10. What is your biggest complaint about the 1960 construction area, if any?

Construction too slow -- 18.1\%
Delay -- 12.9\%
Signs/Barrels -- 10.5\%
12. From the list below, what would you say is the biggest problem in the FM 1960 construction area?
12.6\% - travel delay
13.7\% - the work has taken too long
23.2\% - the construction area (in miles) is too long
.5\% - signs are confusing
9.0\% - too much traffic
17.9\% - difficult to turn
1.1\% - difficult to find where you're going
11.6\% - hazardous road conditions
10.5\% - general confusion

- other (please specify)

2. Was this delay unreasonable?
yes -- $33.3 \%$
no $--66.0 \%$
other --. $7 \%$

3b. If yes, how much are you delayed by the construction during rush hour?
(3 most frequent response percentages)
10 min. -- 12.2\%
20 min. -- 17.0
30 min . -- $26.8 \%$
4. Are you using other routes to get where
you want to go, because of the construction on FM 1960?

$$
\text { yes -- } 85.7 \% \text { no -- } 13.8 \% \text { other -- . } 5 \%
$$

5b. If no, why not?
7. Are there too many signs, too few signs, or the right amount of signs that give warnings and information about how to drive through the construction area?
too many -- 9.4\% too few -- 13.8\%
right amount -- 73.3\% Comments -- $3.5 \%$
9. Should there be more, less, or about the same number of barrels through the construction area?

$$
\text { more -- } 4.9 \% \quad \text { less -- } 21.7 \%
$$

same number .- $70.4 \%$

$$
\begin{gathered}
\text { less -- } 21.7 \% \\
\text { Comments -- } 3.0 \%
\end{gathered}
$$

11. Do you have any other complaints or comments about the 1960 construction area?

Turning problems -- 14.9\%
Too much construction at one time -- $11.9 \%$ Construction too slow -- $10.5 \%$
13. Is there anything you would like to add about construction areas in general, or about the State Highway Department in general?

Compliment to SDHPT -- $28.4 \%$
Work faster -- 13.7\%
Too many roads under construction -- 9.2\% Work zones should be shorter -- 9.2\%
14. Do you prefer roads to have a continuous left turn lane marked by painted lines on the pavement, or medians with turn lanes cut out of them?
continuous left turn lanes -- 49.5\% medians -- 50.5\%

FIGURE 17 Discussion question summary.
18). Respondents were asked whether they favored or opposed this type of signing. A majority ( 53.5 percent) felt that signs showing directions to retail businesses should be allowed in the construction area. Others objected to such signs because they are distracting ( 14.9 percent), confusing ( 3.5 percent), not official signs ( 3.5 percent), and too small ( 1.0 percent). Several of those who favored the signs suggested that retail owners should be allowed to mitigate the disruptive effect of the construction in terms of visibility and accessibility.

## Signs and Messages as Problems

A third objective of the survey was to determine the relative importance of signing and the motorist information system as a source of concern for the users of FM 1960 during the reconstruction activity. Therefore, survey participants were asked if they believed the signing and channeling devices used were adequate. The responses were fairly positive overall73.3 percent said that the right number of barrels had been placed through the construction area.

Figure 19 shows a type of sign developed by the Houston Northwest Chamber of Commerce. The use of nonstandard signing is occasionally justified by particularly sensitive issues, as in the case of the FM 1960 reconstruction. Because these signs seemed to represent an effort to add a certain lightness to messages given to motorists, their effect was measured. A majority of those surveyed ( 66.2 percent) said they like the messages on the circular red signs. In general, drivers interpreted them as positive messages. About 20 percent said they did not like them, and 10.8 percent also said they were either distracting or hazardous.

A brief biographical summary concluded the interview. The results are shown in Figure 20.

## CONCLUSIONS

The survey approach proved to be an effective tool for assessing certain difficulties with sign comprehension in a work zone. The use of a booklet with photographs of various traffic control devices in and out of context produced a sizable num-


FIGURE 18 Sign showing direction to retail business in construction zone.


FIGURE 19 Nonstandard signs used in FM 1960 construction area.
ber of interpretations in a reasonable time frame with minimal effort on the part of the respondent. One weakness of the approach was that the interpretations given for the survey are not necessarily predictors of behavioral responses to the traffic control devices in the roadway environment. Further, in some cases the incorrect interpretation may be of very little negative consequence. An index that would weigh responses for misinterpretation consequences is recommended for future analyses.
The survey of FM 1960 users confirmed previously conducted studies (7) showing that all aspects of signing are not fully understood by motorists. Symbol signs (Advance Flagger, Low Shoulder, and Lane Ends) were correctly interpreted by 77,13 , and 78 percent of the respondents, respectively. Photographs showing the signs in context elicited only slight improvements.

Depending on the situation (in or out of context), from 42 to 33 percent of the drivers surveyed were confused about the meaning of the Advance Construction sign. Further, No Center Lane and No Center Turn Lane are two common signs that do not give the motorist a clear message of the response required.

Inconsistent placement of the Crossover sign (i.e., both in front of and behind the area to be used for crossing over)

| Sex | 47.0 Male | 53.0 Female |  |  |  |
| :---: | :---: | :---: | :---: | :---: | :---: |
| Race | 81.3 Anglo | 7.0 Black | 7.5 | Hispanic | 4.2 Other |
| Age | 15.2 under 25 | 76.0 26-55 | 8.8 | over 55 |  |
| Highest level of education received |  |  |  |  |  |
| 8.9 Less than high school 29.6 Some college |  |  | $\begin{aligned} & 24.1 \\ & 37.4 \end{aligned}$ | High schoo College gra | duate |
| How many years have you been driving? |  |  |  |  |  |
| How many years have you lived in Houston? |  |  |  |  |  |
| What is your zip code? |  |  |  |  |  |
| How often do you travel on FM 1960? |  |  |  |  |  |
| 42.1 at least once daily <br> 14.7 at least once monthly |  |  |  | at least on less than | eekly <br> month |

FIGURE 20 Biographical summary of respondents surveyed.
forces the motorist to rely on cues from the roadway environment. An improvement was found in the continuity of responses to the green Crossover sign when it was posted in combination with a ribbon barrier.

However, sign and message interpretation were not primary sources of concern for users of FM 1960. More important issues involved the length of the project, problems associated with turning, and travel delay. The length of the project (in time) was the most frequently cited (18 percent) personal complaint about the construction, while the length (in miles) was most frequently ( 23 percent) checked from a list of problems. Problems associated with turning and travel delay were checked as the biggest problems by 18 and 13 percent of the respondents, respectively.

The survey instrument focused on problems that might have been encountered by users of FM 1960 during the reconstruction activity. Despite the construction aggravation pointed out in the survey, however, 91 percent of the drivers surveyed believed the long-term benefits will outweigh the short-term inconveniences. In general, the survey respondents indicated a tolerance for construction and its related problems and have positive attitudes toward the Texas State Department of Highways and Public Transportation.

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# Motorist Understanding of and Preferences for Left-Turn Signals 

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A survey of licensed drivers was conducted at the 1988 Indiana State Fair to determine motorists' understanding of and preferences for left-turn signal alternatives including permissive, protected, and both protected and permissive ( $\mathrm{p} / \mathrm{p}$ ) signals, and leading and lagging phase sequences. Survey responses were received from a diverse but generally representative sample of over 400 people. Statistics such as the respondent error rate during the understanding portion of the survey consistently indicated that the survey data were not biased in any substantive way. Several notable results emerged from the analysis of the survey responses, The protected signal was by far the best understood, whereas the $\mathrm{p} / \mathrm{p}$ was the most often misunderstood. The Left Turn Yield on Green sign proved more confusing than the other $p / p$ sign conditions tested, including the no sign condition. Among signals, the protected was most often preferred, and the permissive proved the least popular. For many reasons, the leading sequence was preferred by more respondents than the lagging sequence.

A survey of licensed Indiana drivers was conducted as part of a research effort on the effects of left-turn signal alternatives in Indiana. The purpose of the survey was to determine the relative levels of understanding of and preferences for the various left-turn alternatives under consideration. The results were used with other information gathered during the research to help establish guidelines for the placement of various left-turn signal alternatives.

The following signal alternatives were included in the survey:

- The permissive scheme, under which vehicles may turn left when receiving a green ball signal and when sufficient gaps appear in the opposing traffic stream, which also has a green ball signal;
- The protected scheme, under which vehicles may turn left only when receiving a green arrow signal that affords them the exclusive right-of-way through the intersection, and
- The $\mathrm{p} / \mathrm{p}$ scheme, under which protected left turns may be made at another point in the cycle.

In Indiana, the $\mathrm{p} / \mathrm{p}$ scheme is accomplished most often by the use of a doghouse display with five signal lenses. Also of interest was the question of whether, for protected and $p / p$ schemes, the green arrow phase should precede or follow (lead or lag) the green ball phase.

[^21]Previous surveys ( $1-5$; P. Basha, unpublished memo) have been conducted on the subject of left-turn treatments. However, several reasons prompted the belief that a new survey would provide more worthwhile data. First, the context of the previous surveys, including time and place, was significantly different from that of Indiana in 1988. Second, the respondents to previous surveys came from similar areas, had similar backgrounds, or were limited in number. Finally, although data on the understanding of signal alternatives were plentiful in the literature, data on preferences for different signal alternatives were sparse. Especially critical was the paucity of data on preferences for leading or lagging left-turn phases. Thus, a survey overcoming these limitations was desired.

## METHODS

A survey instrument that would overcome the limitations of previous surveys, provide data relatively quickly, and remain within project budgetary restrictions was desired. After more traditional telephone and mail survey techniques had been explored and rejected because of the very complex messages to be conveyed to respondents, a personal interview format was selected. The 1988 Indiana State Fair was selected as the time and place for the interviews. The state fair provided a convenient forum in which a large, diverse sample of drivers from all parts of the state could answer questions.

The script for the interviews was pilot tested and revised many times before the state fair. The final script contained questions in three major areas: respondent demographic data (i.e., age, sex, county of residence, and number of miles driven per year), understanding of left-turn alternatives, and preferences for left-turn alternatives. For the main part of the understanding portion of the survey, each respondent viewed eight sign and signal displays and was asked to choose the correct action from among four potential left-turn actions. Table 1 presents the eight signal displays each subject viewed during the understanding portion and the four action choices presented with the displays. Table 1 also presents the definitions for correct left-turn actions, close (conservative) errors (actions that would probably not have catastrophic consequences in traffic), and gross errors (actions that would probably result in a catastrophe in traffic) from among the four choices for action for each display. Three sign conditions were tested with each of the three protected signal displays and with each of the three $p / p$ signal displays, as shown in Figure 1. During the preferences portion of the survey, four pairs of

TABLE 1 SIGNAL DISPLAYS, ACTION CHOICES OFFERED, AND ERROR DEFINITIONS FOR THE UNDERSTANDING PORTION OF THE SURVEY

| Display | Choice Number* |  |  |
| :---: | :---: | :---: | :---: |
|  | Correct | Close (Conservative) Error | Gross Error |
| Permissive - red ball | 4 | 3 | 1,2 |
| Permissive - green ball | 2 | 3 | 1,4 |
| Protected - green ball for through, red ball for left | 4 | 3 | 1,2 |
| Protected - green ball for through, green arrow for left | 1 | 2 | 3,4 |
| Protected - red ball for through, green arrow for left | 1 | 2 | 3,4 |
| Protected / Permissive - green ball | 2 | 3 | 1,4 |
| $\begin{gathered} \text { Protected } / \begin{array}{c} \text { Permissive - green ball } \\ \text { for through, green } \\ \text { arrow for left } \end{array} \\ \hline \end{gathered}$ | 1 | 2 | 3,4 |
| Protected / Permissive - red ball for through, green arrow for left | 1 | 2 | 3,4 |

* $1=$ Turn left without stopping because you have the right-of-way.
$2=$ Turn left without stopping unless you must wait for oncoming traffic to clear.
$3=$ Stop. Then, turn left when oncoming traffic clears.
$4=$ Stop. Do not turn until the signal changes to indicate you may proceed.

Protected/Permissive

No Sign
VS.

| LEFT TURN |
| :---: |
| ON |
| GREEN |
| OR |
| ARROW |

vs.


## Protected



FIGURE 1 Sign conditions tested.
signal alternatives (all with no signs) were offered to the respondents, including permissive versus protected, permissive versus $\mathrm{p} / \mathrm{p}$, protected versus $\mathrm{p} / \mathrm{p}$, and leading versus lagging sequences. After viewing a pair of signal alternatives, respondents were asked which alternative they preferred and why, or whether they had no preference for either alternative. Within
the understanding and preferences portions of the survey, the order in which particular displays were shown was randomized to avoid bias.
The displays shown to the respondents while questions were asked were 8.5 - by $11-\mathrm{in}$. black-and-white copies of a drawing of a hypothetical intersection with the appropriate signals or
sign representing the left-turn alternative. The sample display shown in Figure 2 differs from an actual display shown during an interview only in that the active signal lenses were colored (red, yellow, or green). The design of the displays was based on the displays developed for another recent survey (5) of motorist understanding of left-turn signals. The major advantage of the displays was that they conveyed the idea of the left-turn alternative in the context of a typical intersection (a four-lane divided street with left-turn bays meeting a minor street) without distracting background noise, because the main points of the survey were understanding and preference, rather than perception. However, because the displays were static, changes in signal indication were difficult to depict. Figure 3 shows a display developed for the question on preferences for leading or lagging left turns, for which the signal sequence was the main point of the presentation.

The interviews were conducted from 9:00 a.m. to 5:00 p.m. on the first 4 days of the 1988 Indiana State Fair (Wednesday, August 17th through Saturday, August 20th). The state fairgrounds are in Indianapolis, so the fair attracts many people from that metropolitan area. However, the central location of Indianapolis and the wide variety of different exhibits attract many different types of people to the fair from all parts of the state. The interviews were conducted at a table on the second floor of the 4-H Exhibit Hall in an area devoted otherwise to arts and crafts displays and demonstrations. The location proved advantageous, because a steady number of people walked past the table. Also, no particular bias toward traffic or highways was evident in the population of passers-by (as opposed to a location near the Indiana Department of Trans-
portation booth, for example, which might have attracted respondents particularly interested in, or unhappy about, traffic or highways). The booth was adorned with mock Stop signs and traffic signals and posters explaining the general purposes of the survey (traffic signals and safety) and the names of sponsoring organizations.

Respondents were procured in two ways. People walking by the table who took an obvious interest in the posters and signs were asked by survey personnel whether they wished to participate. Most of these people were eager to help with the survey. In addition, interviewers asked each adult passer-by to participate in the survey. This method yielded many respondents, although the nonresponse rate was high. Although statistics on nonresponse were not maintained, survey personnel estimated that about half of the people asked to participate without first expressing an interest refused to do so. The bias introduced to the survey results by these refusals was small, however, because the reasons people gave for not responding had nothing to do with the survey purpose and because the exact survey purpose (i.e., left-turn traffic signals) was not revealed until some expression of interest was shown by a potential respondent.

Respondents received three fair amusement coupons (worth $\$ 0.45$ each) for completing the interview. Interviews lasted 5 to 10 min and were conducted by graduate students in the transportation engineering program in the Purdue School of Civil Engineering. The interviewers were thoroughly briefed before the survey began and were encouraged to repeat the script as closely as possible with each respondent to avoid bias between interviewers.


FIGURE 2 Typical survey display.


FIGURE 3 Lagging and leading sequence display.

## RESULTS

After an initial warm-up period for interviewers on the first day, the survey proceeded without problems or changes. During the four survey days, 402 responses were recorded. All respondents were licensed drivers or holders of learner's permits who claimed an Indiana address.

The survey respondents were representative of the population of Indiana drivers in several ways but differed from that population in several other ways. The most significant way in which the sample was representative of Indiana drivers in general was the distribution of the respondents' residences. The breakdown of reported counties of residence revealed
that responses were received from people living in 85 of the 92 counties in Indiana. The ages reported by respondents also revealed a wide distribution. Table 2, presenting the breakdown of the responses to the question on age, indicates that the most frequent response and the 50th-percentile response was for the 36 - to 45 -year age group and that younger and older drivers were well represented. The reported mileage driven by respondents was also representative of the general population, which was not surprising considering that the question on the subject was worded to mention the average general mileage of $10,000 \mathrm{mi} / \mathrm{year}$. The median and mean numbers of reported annual miles driven were 10,000 and 14,000 , respectively, on a range of 100 to $100,000 \mathrm{mi} /$ year.

TABLE 2 RESPONDENT AGE DISTRIBUTION

| Age Group, <br> Years | Number of <br> Responses | Percent of Total <br> Responses | Percent of <br> Licensed Drivers* |
| :---: | :---: | :---: | :---: |
| $16-25$ | 94 | 23.4 | 21.4 |
| $26-35$ | 84 | 20.9 | 23.8 |
| $36-45$ | 150 | 37.3 | 18.2 |
| $46-55$ | 44 | 10.9 | 13.2 |
| $56-65$ | 22 | 5.5 | 12.6 |
| 66 or over | 8 | 2.0 | 10.8 |
| Total | 402 | 100.0 | 100.0 |

[^22]reports.(7)

Fifty-seven percent of survey respondents were female, whereas 49 percent of licensed drivers in Indiana (in 1984) were female (6).

The survey was not especially representative for the proportion of urban to rural area residents responding. Only 52 percent of the respondents were from urban counties (defined as belonging to Standard Metropolitan Statistical Areas), as opposed to the 1980 statewide population figure of 67 percent (7). The overabundance of rural county residents in the sample was treated by close examination of the urban or rural county of residence variable throughout subsequent analyses. Indiana is a fairly densely and evenly populated state with a wide geographical distribution of left-turn signals. Most rural counties, therefore, contain or are near towns or cities with left-turn signals, and most drivers living in rural counties regularly encounter such signals, so the overabundance of rural respondents was not considered especially critical. In sum, although the survey sample included higher proportions of female and rural drivers than the general Indiana population, the sample provided a good representation of the population for a survey gathered in one place over a limited time.

## Error Rate

The quality of the responses to the survey was judged partially by an analysis of the error rate on the questions testing motorist understanding. Table 3, which presents the number of errors (i.e., incorrect responses of any type) made by the respondents on the nine questions in the understanding portion of the survey, indicates that the number of errors was well distributed. Few people entirely misunderstood the survey methodology or displays, because only one person got all nine questions wrong and only 18 people got seven or more questions wrong. Table 3 also indicates that the survey ques-
tions were not too easy, because only 43 respondents gave correct responses for all nine questions. Because most respondents made errors on a few questions, differences between displays were probably the cause of respondents' errors, as had been hoped, rather than flaws in the survey methodology.

The error rate on the nine understanding questions was analyzed with other variables to see whether patterns of errors emerged. Of special interest was the relationship between the error rate on the nine understanding questions and the particular interviewer, and between the error rate and the day the interview was conducted. Using SAS (8) to compute the chi-squared value as a test of the degree of association between the error rate and the particular interviewer, the significance probability ( $p$ ) was found to be 0.989 , indicating that the two variables were not related at the 0.05 level of significance. The chi-squared significance probability for the association between the error rate and the day of the interview was 0.954 , indicating that those two variables were not closely related either. Both of these findings lend credence to the view that the quality of the survey data was high.
The error rate was also tabulated with respondent characteristics, including age, sex, annual miles driven, and urban or rural county of residence. The resulting significance probabilities were 0.356 with the age variable, 0.299 with the sex variable, 0.234 with the annual miles driven variable, and 0.079 with the urban or rural county of residence variable. None of the variables were significantly associated with the error rate at the 0.05 level, although the urban or rural county of residence variable probability was near 0.05 . Table 4 indicates that urban county residents made slightly fewer errors than rural county residents. Because urban county residents were underrepresented in the survey sample, the error rate of the entire population of Indiana drivers (if they all took the survey) would probably be slightly lower than the error rate of the survey sample.

TABLE 3 DISTRIBUTION OF NUMBERS OF ERRORS ON NINE UNDERSTANDING QUESTIONS

| Number of <br> Errors | Number of <br> Respondents | Percent of <br> Total |
| :---: | :---: | :---: |
| 0 | 43 | 10.7 |
| 1 | 52 | 12.9 |
| 2 | 68 | 16.9 |
| 3 | 88 | 21.9 |
| 4 | 58 | 14.4 |
| 5 | 46 | 11.4 |
| 6 | 29 | 7.2 |
| 7 | 9 | 2.2 |
| 8 | 8 | 2.0 |
| 9 | 1 | 0.2 |
| Total | 402 | 100.0 |

TABLE 4 DISTRIBUTION OF NUMBERS OF ERRORS ON NINE
UNDERSTANDING QUESTIONS BY URBAN VERSUS RURAL COUNTIES OF RESIDENCE

| Number of <br> Errors | Urban County <br> Respondents | Rural County <br> Respondents | Total <br> Respondents |
| :---: | :---: | :---: | :---: |
| 0 | 25 | 18 | 43 |
| 1 | 29 | 23 | 52 |
| 2 | 40 | 28 | 68 |
| 3 | 49 | 39 | 88 |
| 4 | 23 | 35 | 58 |
| 5 | 26 | 20 | 46 |
| 6 or more | 17 | 30 | 47 |
| Total | 209 | 193 | 402 |

## Understanding and Sign Condition

The results for the understanding portion of the survey regarding signing conditions are presented in Tables 5 and 6 for the six signal displays that had variable signing conditions. The
results for the protected signal displays in Table 5 indicate that no particular pattern was prevalent for the relative understanding of the no sign condition, the Left Turn on Arrow Only sign, and the Left Turn Signal sign. Even for the simultaneous green ball and green arrow display, which boasts a

TABLE 5 UNDERSTANDING OF SIGN DISPLAY ALTERNATIVES FOR A
PROTECTED SIGNAL

| Signal Display | Sign Display | Response Class |  |  |  | p -value |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: |
|  |  | Correct <br> Responses | Close | Gross <br> Errors | Total <br> Responses |  |
| Green Ball for Through Traffic, Red Ball for Left Turns | No Sign | 125 | 8 | 2 | 135 | 0.504* |
|  | $\begin{aligned} & \text { "Left Turn } \\ & \text { on Arrow } \\ & \text { Only" } \\ & \hline \end{aligned}$ | 126 | 5 | 2 | 133 |  |
|  | "Left Turn Signal" | 122 | 6 | 6 | 134 |  |
|  | Total | 373 | 19 | 10 | 402 |  |
| Green Ball for Through Traffic, Green Anrow for Left Turns | No Sign | 97 | 29 | 9 | 135 | 0.022 |
|  | $\begin{aligned} & \text { "Left Turn } \\ & \text { on Arrow } \\ & \text { Only" } \end{aligned}$ | 97 | 19 | 17 | 133 |  |
|  | "Left Turn Signal" | 86 | 39 | 9 | 134 |  |
|  | Total | 280 | 87 | 35 | 402 |  |
| Red Ball for Through Traffic, Green Arrow for Left Turns | No Sign | 99 | 24 | 12 | 135 | 0.173 |
|  | $\begin{aligned} & \text { "Left Turn } \\ & \text { on Arrow } \\ & \text { Only" } \end{aligned}$ | 102 | 14 | 17 | 133 |  |
|  | "Left Turn Signal" | 103 | 23 | 8 | 134 |  |
|  | Total | 304 | 61 | 37 | 402 |  |

* For a chi-square analysis in which the close (conservative) error and gross error columns were combined.

TABLE 6 UNDERSTANDING OF SIGN DISPLAY ALTERNATIVES FOR A p/p SIGNAL

| Signal Display | Sign Display | Response Class |  |  |  | p-value |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: |
|  |  | Correct <br> Responses | Close <br> Errors | Gross <br> Errors | Total Responses |  |
| Green Ball for Through Traffic and Left Tums | No Sign | 54 | 50 | 30 | 134 | 0.213 |
|  | $\begin{aligned} & \text { "Left Turn } \\ & \text { on Green } \\ & \text { or Arrow" } \\ & \hline \end{aligned}$ | 68 | 33 | 34 | 135 |  |
|  | "Left Turn Yield on Green | 58 | 46 | 29 | 133 |  |
|  | Total | 180 | 129 | 93 | 402 |  |
| Green Ball <br> for Through <br> Traffic, <br> Green <br> Arrow for <br> Left Turns | No Sign | 88 | 36 | 10 | 134 | < 0.0005 |
|  | $\begin{aligned} & \text { "Left Turn } \\ & \text { on Green } \\ & \text { or Arrow" } \end{aligned}$ | 93 | 27 | 15 | 135 |  |
|  | "Left Turn Yield on Green | 56 | 47 | 30 | 133 |  |
|  | Total | 237 | 110 | 55 | 402 |  |
| Red Ball for Through Traffic, Green Arrow for Left Tums | No Sign | 80 | 28 | 26 | 134 | 0.026 |
|  | "Left Turn on Green or Arrow" | 92 | 16 | 27 | 135 |  |
|  | Left Turn Yield on Green | 71 | 37 | 25 | 133 |  |
|  | Total | 243 | 81 | 78 | 402 |  |

chi-squared significance probability of 0.022 (indicating a significant relationship at the 0.05 level) having no sign was just slightly superior to the other sign conditions, and little distinguished the performance of the Left Turn on Arrow Only sign from the performance of the Left Turn Signal sign. From Table 6 for the $p / p$ signal displays, a clear pattern emerges. The Left Turn Yield on Green or Arrow sign performed better than no sign and much better than the Left Turn Yield on Green e sign. The latter sign was associated with far fewer correct answers, far more conservative errors, and far more gross errors of understanding than the other two signing conditions for $\mathrm{p} / \mathrm{p}$ signals, when a green ball for through traffic and a green arrow for left turns were displayed. An analysis of the signing conditions using data only from survey respondents from urban counties revealed trends similar to those for the full data set.

## Understanding of Signals

The understanding portion of the survey was analyzed using four comparisons of the relative understanding of different signal schemes. Tables $7-10$ present the data and statistical
test results for these four comparisons. Table 7 indicates that the permissive and $p / p$ signal schemes, when both were displayed with green ball signals, generated almost identical numbers of correct responses. The permissive scheme, however, had a significantly greater proportion of conservative errors at the 0.05 level using the Z-test for proportions (9) and a correspondingly smaller number of gross errors. Table 8 indicates that the protected scheme inspired a significantly greater number of correct responses than the permissive scheme when both were displayed with red ball signals. For displays with a green left-turn arrow and green ball signals for through traffic (Table 9) and a green left-turn arrow and red ball signals for through traffic ('I'able 10), the protected signal scheme had significantly more correct responses, significantly fewer gross errors, and marginally fewer conservative errors than the $\mathrm{p} / \mathrm{p}$ scheme. From these results, the relative levels of understanding of the signal schemes tested are clear: protected signals were the best understood, permissive signals were less well understood, and $p / p$ signals were the least understood. The four comparisons presented in Tables 7-10 were also made using data exclusively from urban county residents, but the results were generally no different.

TABLE 7 RELATIVE UNDERSTANDING OF PERMISSIVE AND p/p SIGNALS WHEN A GREEN BALL (ONLY) IS DISPLAYED

| Response Class | Signal | Number of Responses | Proportion of (402) Responses | Z <br> Computed | Significant Difference at 0.05 Level? |
| :---: | :---: | :---: | :---: | :---: | :---: |
| Correct | Permissive | 181 | 0.450 | 0.06 | No |
|  | Protected / Permissive | 180 | 0.448 |  |  |
| Close (conservative) Ertor | Permissive | 179 | 0.445 | 3.70 | Yes |
|  | Protected / Permissive | 128 | 0.318 |  |  |
| Gross Error | Permissive | 42 | 0.104 | 4.60 | Yes |
|  | Protected / Permissive | 94 | 0.234 |  |  |

TABLE 8 RELATIVE UNDERSTANDING OF PERMISSIVE AND PROTECTED SIGNALS WHEN A RED BALL (ONLY) IS DISPLAYED

| Response <br> Class | Signal | Number of <br> Responses | Proportion <br> of (402) <br> Responses | Z <br> Computed | Significant <br> Difference at <br> 0.05 Level? |
| :---: | :---: | :---: | :---: | :---: | :---: |
| Correct | Permissive | 336 | 0.836 | 4.04 | Yes |
|  | Protected | 373 | 0.928 |  | 4.39 |
| Close <br> (conservative) <br> Error | Permissive | 55 | 0.137 | Yes |  |
| Gross <br> Eror | Protected | 19 | 0.047 | Permissive | 11 |
|  | Protected | 10 | 0.027 | 0.22 | No |

TABLE 9 RELATIVE UNDERSTANDING OF PROTECTED AND p/p SIGNALS WHEN A GREEN BALL FOR THROUGH TRAFFIC AND A GREEN ARROW FOR LEFT TURNS ARE DISPLAYED

| Response Class | Signal | Number of Responses | Proportion of (402) Responses | $\mathbf{Z}$ <br> Computed | Significant Difference at 0.05 Level? |
| :---: | :---: | :---: | :---: | :---: | :---: |
| Correct | Protected | 280 | 0.696 | 3.15 | Yes |
|  | Protected / Permissive | 237 | 0.590 |  |  |
| $\begin{array}{\|c\|} \hline \text { Close } \\ \text { (conservative) } \\ \text { Error } \\ \hline \end{array}$ | Protected | 87 | 0.216 | 1.89 | No |
|  | Protected / Permissive | 110 | 0.274 |  |  |
| Gross Error | Protected | 35 | 0.087 | 2.23 | Yes |
|  | Protected / Permissive | 55 | 0.137 |  |  |

The data from the understanding portion of the survey were also examined to determine which signal phases for the protected, $\mathrm{p} / \mathrm{p}$, and permissive signals were most misunderstood. Tables 7 and 8 indicate that the green ball phase for the permissive signal was far more often misunderstood ( 181 correct responses) than the red ball phase ( 336 correct responses). Tables $8-10$ indicate that the protected signal most often inspired a correct response when respondents viewed a red
ball ( 373 correct responses), whereas the difference between the other two phases tested was not significant (the green arrow with red ball had 304 correct responses, and the green arrow with green ball had 280 correct responses). Finally, although none of the three phases of the $\mathrm{p} / \mathrm{p}$ signal tested generated a high number of correct responses, the green ball phase (Table 7, 180 correct responses) was the most misunderstood. The green arrow with red ball phase of the $\mathrm{p} / \mathrm{p}$

TABLE 10 RELATIVE UNDERSTANDING OF PROTECTED AND p/p SIGNALS WHEN A RED BALL FOR THROUGH TRAFFIC AND A GREEN ARROW FOR LEFT TURNS ARE DISPLAYED

| Response Class | Signal | Number of Responses | Proportion of (402) Responses | $\begin{gathered} \mathrm{Z} \\ \text { Computed } \end{gathered}$ | Significant Difference at 0.05 Level? |
| :---: | :---: | :---: | :---: | :---: | :---: |
| Correct | Protected | 304 | 0.756 | 4.61 | Yes |
|  | Protected / Permissive | 243 | 0.604 |  |  |
| Close (conservative) Error | Protected | 61 | 0.152 | 1.85 | No |
|  | Protected / Permissive | 81 | 0.202 |  |  |
| Gross <br> Eror | Protected | 37 | 0.092 | 4.13 | Yes |
|  | Protected / Permissive | 78 | 0.194 |  |  |

signal (Table 10) had about the same number of correct responses as the green arrow with green ball phase (Table 9), but because the green arrow with red ball phase also had significantly more gross errors ( 78 to 55 ), this signal phase should be considered the more misunderstood of the two on the basis of these survey data.

## Preferences for Signal Alternatives

A summary of survey responses to the questions on driver preferences for left-turn signals is presented in Table 11. Those data indicate that the protected signal was clearly preferred over the permissive and $p / p$ signals, the $p / p$ signal was preferred by more respondents than the permissive signal, and the leading signal sequence was preferred more often than the lagging sequence. For all the comparisons in Table 11, 95 percent confidence intervals on the proportion of respondents choosing one or the other signal alternative (9) lie outside 0.5 , meaning that the differences expressed between alter-
natives are all significant at the 0.05 level. The preference for leading over lagging sequences was not as strong as the confidence interval would indicate, though, because almost 100 respondents had no preference.

A summary of the breakdown of preference responses, presented in Table 12, indicates that most of the preference results were unrelated to the variables examined. Age was found to be related to the preference of protected or $\mathrm{p} / \mathrm{p}$ signals, with people in the 16 - to 25 -year group preferring a $\mathrm{p} / \mathrm{p}$ signal more often. Age was related ( $p=0.054$ ) to preference of leading or lagging sequence, although the main contributor to the high chi-square value in this case was the tendency of younger drivers to have no preference more often. The preference for protected or $\mathrm{p} / \mathrm{p}$ signals was related ( $p=$ 0.060 ) to the annual miles driven, with respondents driving the least showing greater preference for $\mathrm{p} / \mathrm{p}$ signals. The annual miles driven variable was also related $(p=0.056)$ to the preference for leading or lagging signals, with the people driving the least opting for the lagging sequence or the no-preference alternatives more often. Finally, the particular interviewer

TABLE 11 PREFERENCE QUESTIONS SUMMARY

| Signal <br> Alternatives | Number of Respondents Expressing a Preference | Respondents Preferring Alternative |  | Confidence Interval(0.05 level) |  |
| :---: | :---: | :---: | :---: | :---: | :---: |
|  |  | Number | Proportion | Lower Limit | Upper Limit |
| Protected | 391 | 382 | 0.977 | 0.962 | 0.992 |
| Permissive |  | 9 | 0.023 | 0.008 | 0.038 |
| Protected | 364 | 312 | 0.857 | 0.821 | 0.893 |
| Protected / Permissive |  | 52 | 0.143 | 0.107 | 0.179 |
| Permissive | 376 | 39 | 0.104 | 0.073 | 0.135 |
| Protected / Permissive |  | 337 | 0.896 | 0.865 | 0.927 |
| Leading | 307 | 248 | 0.808 | 0.764 | 0.852 |
| Lagging |  | 59 | 0.192 | 0.148 | 0.236 |

TABLE 12 RELATIONSHIPS BETWEEN PREFERENCES FOR SIGNAL ALTERNATIVES AND VARIOUS INDEPENDENT VARIABLES (EXPRESSED AS CHI-SQUARED SIGNIFICANCE PROPORTION)

| Variable Preference Question   <br>  Protected <br> vs. <br> Protected / <br> Permissive* Permissive <br> v. <br> Protected / <br> Permissive*  | Leading <br> vs. <br> Lagging |  |  |
| :--- | :---: | :---: | :---: |
|  | $<0.0005$ | 0.240 | 0.054 |
| Sex | 0.224 | 0.704 | 0.126 |
| Urban or Rural County of Residence | 0.500 | 0.848 | 0.002 |
| Annual Miles Driven | 0.060 | 0.791 | 0.056 |
| Interviewer | 0.293 | 0.779 | 0.019 |
| Day of Interview | 0.493 | 0.295 | 0.224 |
| Number of Errors on Nine Understanding | 0.140 | 0.394 | 0.526 |
| Questions |  |  |  |

* Chi-square values were calculated from tables which did not include "no preference" responses.
was found to be related to the results for the leading or lagging question. Fortunately, the trend that emerged in this relationship involved two interviewers, one who recorded a sizable number of no-preference responses and another who recorded very few no-preference responses, so the data for the leading and lagging sequences themselves did not depend on particular interviewers. The quality of the survey is reflected in the fact that the interviewer was unrelated to the results for the other questions shown in Table 12 and that the day on which a particular interview was conducted was unrelated to the results for all the preference questions.

Table 12 presents the relationship between the urban and rural county of residence variable and the preferences expressed for various signal alternatives. The preferences expressed for protected or $\mathrm{p} / \mathrm{p}$ signals and the preferences expressed for permissive or $\mathrm{p} / \mathrm{p}$ signals were not significantly related to county
of residence. Respondents from urban counties preferred protected over $\mathrm{p} / \mathrm{p}$ and $\mathrm{p} / \mathrm{p}$ over permissive signals in about the same proportions as residents of rural counties. The preference expressed for leading or lagging sequence was significantly related to county of residence. Table 13 indicates that a much higher proportion of rural county residents preferred lagging to leading sequences than urban county residents. Since urban county residents were underrepresented in the respondent sample, the proportion of the total Indiana driver population that prefers leading sequences is probably higher than that reported in Table 11 for the full survey sample.

Table 12 also presents the relationship between the number of errors on the understanding questions and the preferences expressed for the various signal alternatives. The tabulated error rates were clearly unrelated to the preferences expressed. Preferences expressed by respondents who demonstrated good

TABLE 13 PREFERENCES FOR LEADING OR LAGGING SIGNAL PHASE SEQUENCES BY URBAN VERSUS RURAL COUNTY OF RESIDENCE

| County of <br> Residence | Preference |  |  |  |
| :---: | :---: | :---: | :---: | :---: |
|  | Leading | Lagging | No <br> Preference | Total |
|  | 140 | 18 | 51 | 209 |
| Rural | 108 | 41 | 44 | 193 |
| Total | 248 | 59 | 95 | 402 |

TABLE 14 SUMMARY OF RESPONDENTS CITING VARIOUS REASONS FOR EXPRESSED SIGNAL PREFERENCES

| Preference | Reason |  |  |  |  |
| :---: | :---: | :---: | :---: | :---: | :---: |
|  | Safer | Less Delay | Less <br> Confusion | More Like Normal | Unsure or Other |
| Protected vs. Permissive | $\begin{gathered} 69 \\ 0 \end{gathered}$ | $\begin{array}{r} 52 \\ 3 \\ \hline \end{array}$ | $\begin{gathered} 276 \\ 4 \end{gathered}$ | $8$ | $8$ |
| Protected vs. Protected /Permissive | $\begin{aligned} & 8 \\ & 2 \\ & \hline \end{aligned}$ | $\begin{array}{r} 5 \\ 17 \\ \hline \end{array}$ | $\begin{gathered} 280 \\ 21 \end{gathered}$ | $\begin{gathered} 11 \\ 5 \end{gathered}$ | $\begin{aligned} & 12 \\ & 10 \end{aligned}$ |
| Protected / Permissive vs. Permissive | $\begin{array}{r} 50 \\ 0 \\ \hline \end{array}$ | $\begin{array}{r} 59 \\ 2 \\ \hline \end{array}$ | $\begin{gathered} 229 \\ 31 \\ \hline \end{gathered}$ | $\begin{gathered} 13 \\ 1 \\ \hline \end{gathered}$ | $\begin{gathered} 12 \\ 5 \\ \hline \end{gathered}$ |
| Leading vs. Lagging | 61 11 | 65 17 | 27 11 | 73 10 | 39 11 |

understanding of the left-turn signal displays were very similar to preferences expressed by respondents who demonstrated poor understanding of the displays.

The reasons for preferences expressed by respondents were coded by the interviewers, and a summary of the data is presented in Table 14. Respondents overwhelmingly credited the protected signal with causing less confusion when they expressed a preference for it over both the permissive and the $\mathrm{p} / \mathrm{p}$ signal. The protected signal was also preferred over the permissive signal by many respondents because it was perceived as safer and as causing less delay. Reasons given by respondents for preferring $\mathrm{p} / \mathrm{p}$ over permissive signals followed a very similar pattern, with less confusion given predominantly and safer and less delay given by some. The reasons respondents gave for preferring leading over lagging sequences were well distributed, with roughly equal numbers of respondents stating that leading sequences were more like normal (i.e., more common), safer, and associated with less delay.

## SUMMARY

The survey of Indiana drivers conducted at the 1988 Indiana State Fair provided usable results on the understanding of and preferences for various left-turn signal alternatives. Despite the fact that the survey was conducted in one place over a 4 day span, responses were received from a wide variety of people. The error rate computed for the understanding questions, and the lack of association between preferences expressed and particular interviewers or survey days, showed that the survey script, displays, and format were reasonable and that the data were not biased in any substantive way. However, applications of the survey data outside this project must be made carefully with the context of the survey (e.g., the ten-
dencies of Indiana drivers and highways in 1988 and the four-lane boulevard shown in the survey displays) in mind.

Several previously cited results are particularly notable. The protected signal was by far the best understood, and the $\mathrm{p} / \mathrm{p}$ signal was the least understood. The Left Turn Yield on Green - sign proved more confusing than the other $p / p$ signing alternatives tested, but there was little to distinguish the protected signal signing alternatives tested. The protected signal was the most preferred signal because most respondents associated it with less confusion, whereas the permissive signal was least preferred. For a wide variety of reasons, respondents expressed a greater preference for the leading over the lagging sequence.

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# Motorist Information Systems and Recurrent Traffic Congestion: Sensitivity Analysis of Expected Results 

Haris N. Koutsopoulos and Tsippy Lotan


#### Abstract

Urban traffic flows have increased dramatically in recent years, causing alarmingly high levels of congestion. A widely held belief is that the construction of new facilities alone will be unable to alleviate this congestion. In this context, motorist information systems based on modern information technology may play an important role in reducing traffic congestion and improving traffic flows and safety. A methodology that is based on a stochastic traffic assignment model is proposed for assessing the effectiveness of motorist information systems in reducing recurrent traffic congestion and for examining the interactions among important parameters of the problem such as level and amount of information provided, percentage of users and access to information, and congestion levels. The methodology is applied to a small suburban network.


Urban traffic flows have increased dramatically in recent years, causing alarmingly high levels of congestion. In 1985, 61 percent of rush-hour traffic on urban Interstates was rated as congested compared with 40 percent in 1975. The number of cars owned in the United States tripled between 1960 and 1986 and the total annual vehicle-miles increased from 719 to 1,861 billion in the same period. A widely held belief is that the construction of new facilities alone will be unable to alleviate this congestion. Therefore, planners are looking to improve the use of existing facilities with improved traffic management schemes.

Various authors, such as Jeffery (1,2) and Kobayashi (3), indicate that a certain percentage of urban trips are poorly designed and result in unnecessary delays. Hence, the potential exists to improve traffic conditions on urban and freeway networks by using information technology. Better use of existing facilities may be realized by providing information to motorists with respect to alternative paths to their destinations and actual traffic conditions on links of interest using a combination of sideways signals and on-board systems. In this context, motorist information systems that are based on modern information technology may play an important role in reducing traffic congestion and improving traffic flows and safety.

Various methods exist for relaying information to users, each with substantially different technical requirements, implementation costs, complexity, and capabilities in providing information. Examples of motorist information systems

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include pretrip information systems, roadside displays, traffic information broadcasting systems, and electronic route guidance systems. The Bureau of Public Roads (BPR) of the U.S. Department of Transportation is credited with the first attempt at developing a comprehensive electronic route guidance system in 1967 (4). The project, which terminated prematurely in 1970, concluded that systems that provide real time traffic information are promising, but more research is needed to overcome significant challenges in hardware and software.

However, the past few years has been a period of advancement and development in computer, communications, and general information technology. Main characteristics of these technological advances are reduced costs (capital and operating) and increased capabilities making possible efficient collecting and processing of large amounts of data for detailed and sophisticated analysis and control. These advances in information technology, along with the pressures arising from increased congestion, have spurred renewed interest in motorist information systems. Various projects (AUTOGUIDE in England, ALI-SCOUT in Germany, AMTICS in Japan, and PATHFINDER in the United States) demonstrating the feasibility of these systems a:e currently under way.

The hypothesis of the research presented is that provision of information to motorists can reduce traffic congestion, but that there are diminishing marginal returns as more information is provided. In testing this hypothesis, interactions among various important parameters of the problem are examined. These interactions are important because understanding the interactions among parameters will help determine the most appropriate hardware configuration for obtaining and transmitting information and the identification of conditions under which motorist information systems may be most effective.

## RELATED STUDIES

Kobayashi (3) presents some results from a feasibility study of the comprehensive automobile traffic control system (CACS) project. He developed a simulation model to examine the effectiveness of alternative guidance methods and applied the model using data from a pilot area in Tokyo. Factors such as road length, number of lanes, and number of left and right turns were assumed to be the most important attributes that
affect route choice for unguided vehicles, whereas shortestpath criteria were used for route selection by the guided vehicles. The results indicate that total travel time in Tokyo, with the introduction of motorist information systems, could be reduced by 6 percent and fuel consumption by 5 percent.
Jeffery ( 1 ), on the basis of an analysis of times and distances involved in a sample of journeys made in the United Kingdom, suggests that about 2 percent of all driver journey costs, in principle, could be recovered by an efficient route guidance system on the basis of historical information (journey costs include fuel costs, other vehicle running costs, and driver's time).
Tsuji et al. (5) investigated the effectiveness of route guidance systems by using a mathematical model in a case study in the Tokyo area. Measures of effectiveness used were the probability that guided vehicles arrive at their destination before unguided vehicles and the percentage of travel time reduction for guided vehicles. On the basis of this model, guided vehicles arrived at their destination earlier with probability 0.85 and experienced an 11 percent reduction in travel time. However, to be able to apply this model to real data several simplifying assumptions were made, including

1. Flow of guided vehicles does not affect the remaining traffic flow,
2. Travel times on alternative routes are mutually independent,
3. Only two alternative routes exist and each route is used only by the guided vehicles or only by the unguided vehicles,
4. Travel time and predicted travel time are normally distributed and are independent, and
5. All users follow route recommendations.

As a result, the suggested model suffers from several draw-backs-the model is not sensitive to the type of information transmitted to users, does not include interactions between guided and unguided vehicles, and is insensitive to the percent of guided vehicles.

Jeffery (2) combined the results reported by Tsuji et al. (5) with earlier results on the effectiveness of static motorist information systems (1) and concluded that drivers who have cars appropriately equipped could realize, on the average, benefits up to 10 percent if route guidance systems operating in real time are implemented.

Al-Deek et al, (6) estimated potential benefits of in-vehicle information systems in the context of the PATHFINDER project. They conducted a survey to identify routes typically used by commuters in a portion of the SMART corridor in Los Angeles. Traffic along that corridor was simulated and FREQ8PC and TRANSYT-7F were used to determine travel times on the network links. For a combination of four origin and three destination intersections, the cost of different routes was estimated on the basis of shortest path, freeway-biased route, and arterial-biased route. The study assumed that drivers with perfect traffic information would follow the shortest path to their destination. On the basis of these assumptions, savings in travel time estimates based on the differences between the shortest path and other paths are insignificant (less than 3 min for a 20 - to $25-\mathrm{min}$ trip) for the case of recurring congestion.
Under the assumption that an incident on the freeway does not affect travel times on surface streets, Al-Deek et al. (6)
also estimated reduction in travel times under conditions of incident congestion. Under these conditions, maximum savings of 10 min may be realized for a $30-\mathrm{min}$ trip.

Jones et al. (7) addressed the existence of opportunities to improve traffic conditions by providing real-time information to motorists. Data on travel times were collected along three alternative routes for different departure times for a given origin-destination pair on a corridor in Austin, Texas. On the basis of the differences in travel times along these routes, travel times may be reduced by 15 to 30 percent through route change and by 10 to 22 percent through departure time switching.

Gartner and Reiss (8) examined, through simulation, an approach to traffic control in a corridor. This approach incorporated roadside motorist information systems and dynamic setting of traffic signals and ramp metering. Parameters for this integrated system, which was tested in Long Island, New York, were set so that some user-specified performance function is optimized (on the basis of predicted corridor usage). Results indicated small improvements when no accident occurred in the corridor and more substantial benefits, especially with respect to delays and queue length, in the presence of incidents. However, the contribution of the motorist information component of the system (as opposed to the contribution of the dynamic setting of signal and ramp metering parameters) was not clear in this study. Furthermore, the applicability of the conclusion drawn by this study to more general networks was also not clear.

The results of previous studies with respect to the potential role of motorist information systems in reducing congestion and travel times are promising. However, in most cases these results were specific and limited to the data they were generated from while ignoring system-wide impacts and the important interactions among the users of the system. Furthermore, these results provide little insight to the sensitivity of the expected benefits to the parameters and characteristics of the problem.

## METHODOLOGY

Detailed simulation models are necessary for accurate evaluation of the expected benefits from motorist information systems and for fully assessing their potential. A macroscopic model is presented that provides a good understanding of the interactions among the various parameters of the problem and facilitates sensitivity analysis of the expected benefits. However, this model does not account for all the details.

## Characteristics of the Problem

Parameters recognized as important in the application of motorist information systems include users of traffic systems and their behavior, system objectives, congestion characteristics, characteristics of information provided, and network characteristics.

## Motorists

Traditionally, traffic flows in urban networks have been the subject of mandatory control policies, such as traffic signal
systems, which aim at improving system throughput and safety. However, motorist information systems differ from traffic control devices because these systems are a passive form of traffic control. Users receive information about traffic conditions and recommended paths, but are not forced to follow the suggestions. Decisions are made on the basis of perceptions and information received. Consequently, traffic systems are used by a large number of individual units that act independently of each other with each unit having objectives. Hence, in studying motorist information systems the reaction of the users to the information provided needs to be considered. Other important considerations related to the users of urban networks include trip purpose, trip length, degree of user familiarity with the network, and the portion of the motorist population receiving traffic information.

## System Objectives

Reduction of congestion, through better use of existing facilities, is the main system objective in providing traffic information to motorists. However, obtaining system-optimal conditions may conflict with individual user objectives. Therefore, information provided may either reflect actual traffic conditions accurately or be altered to indirectly force users to move toward system-optimal (as opposed to user-optimal) routing decisions. Hence, depending on system objectives, information to motorists may be used as a control variable to influence flow distribution toward ideal levels.

## Congestion Characteristics

Congestion is characterized either as recurrent or incident congestion. Recurrent congestion is caused by the fact that the capacity of certain facilities is inadequate to serve the number of vehicles that want to use the particular link or facility. Incident congestion is caused by incidents that routinely occur in the highway system, for example car breakdown or accidents, that temporarily reduce the capacity of a link or a facility. Various studies (9) indicate that the contributions of recurrent congestion and incident congestion to total traffic delays are similar.

## Motorist Information System Characteristics

An identification and examination of the basic characteristics of motorist information systems is important, especially with respect to services provided and effectiveness in contributing to better motorist decision making. The most important charactcristics are (a) the ability to provide real time infonnation; (b) type, level, extent, and timing of information provided; and (c) ability to address individual vehicles or groups of vehicles.

A basic distinction with respect to the ability to provide real time information is static versus dynamic systems. Dynamic systems provide real time information on the basis of current traffic conditions, whereas static systems only provide historic information. Therefore, only dynamic systems are useful in warning users about incident location and sever-
ity. Static systems are not effective in reducing incident congestion.

Information systems vary from descriptive to prescriptive. Descriptive systems provide information on certain links, or the majority of links around a corridor of interest, whereas prescriptive systems make suggestions on the best available route to follow. Furthermore, (dynamic) systems may provide information only at the beginning of the trip (pretrip information or trip planning scrvices) or may bc capablc of providing updated information throughout the duration of the trip. The determination of how much information, as well as the type of information, to provide has important implications on the design of routing algorithms necessary for the operation of motorist information systems. For example, all users should not choose the same route on the basis of information provided because the results will be opposite to those desired.

The number of users that receive the same information at the same time is an important characteristic that distinguishes different types of motorist information systems. Systems that are capable of providing information tailored to the needs of a request from an individual vehicle are more flexible but require more sophisticated hardware and software requirements for effective operation.

## Network Characteristics

Effectiveness of motorist information systems is affected by the type of physical network and associated traffic levels. Corridors consisting of freeways and arteries with excess total capacity and availability of alternative paths are potentially good candidates for the installation and successful operation of such systems.

## Modeling Approach

The approach selected focuses on evaluating the expected benefits of motorist information systems on recurrent congestion only. In addition, a better understanding is provided of the interactions among the various parameters of the problem: level, amount, and extent of information; number of users who receive information; and congestion levels. In determining these interactions, a hypothesis was made that motorists who receive information on traffic conditions may alter their traffic patterns in the long run. On the basis of information provided continuously, users may realize, for example, that certain links or paths perform consistently better than the ones regularly used, and eventually may switch to those paths.

In order to proceed with development of the necessary models, an assumption was made that users choose routes to minimize some measure of cost (for example, travel time). However, depending on the information available, users have imperfect knowledge of the actual costs associated with alternative paths (links). Hence, decisions on which route to use are made on the basis of perceptions about travel times on various links in the network (and the relative attractiveness of alternative paths to their destination). Furthermore, available traffic information directly affects the perception users of the urban network have of the relative attractiveness of alternative paths to destinations. Hence, perceived travel times
are modeled as random variables whose distribution is influenced by the available information on traffic conditions. Lack of information results in high travel time variance, whereas complete information results in deterministic travel times.

In light of this assumption, the problem can now be placed naturally into the more general class of traffic assignment problems $(10,11)$. In general, two classes of models exist for traffic assignment: deterministic user equilibrium (DUE) and stochastic user equilibrium (SUE).

Deterministic traffic assignment models assume that travel times on the links of the network are deterministic and that users have exact knowledge of travel times and flows on every link. Therefore, these models can be used to represent the case where users receive perfect information from the motorist information system. Equilibrium conditions for DUE can be stated as follows: at DUE, no motorist can improve travel time by unilaterally changing routes.

Stochastic assignment models assume that users have different perceptions on costs along links (or paths) and make decisions (mainly route choice) on the basis of these perceptions. Differences in perceived costs among users are because of different perceptions of path characteristics and various levels of knowledge among users. On the basis of this assumption, perceived travel time on Link $a$ is represented as
$t_{a}=t_{a}\left(x_{a}\right)+\varepsilon_{a}$
where $t_{a}\left(x_{a}\right)$ is the link performance function (i.e., link travel time as a function of the flow $x_{a}$ on the link) and $\varepsilon_{a}$ the perception error with expected value $E\left(\varepsilon_{a}\right)=0$. Thus, cost $C_{k}$ on Path $k$ can be expressed as
$C_{k}=\Sigma t_{a} \delta_{a, k}$
where $C_{k}$ corresponds to perceived travel time on Path $k$ and $\delta_{a, k}$ is equal to 1 if Link $a$ is on Path $k$ and 0 otherwise.

Equilibrium conditions of SUE can be stated as follows: at SUE, no motorist can improve perceived travel time by unilaterally changing routes. Thus, the stochastic equilibrium conditions for a given origin-destination pair are given by
$f_{k}=q p_{k} \quad$ for all Paths $k$
where $q$ is the origin-destination flow, $f_{k}$ is the flow on Path $k$ between the given origin-destination pair, and $p_{k}$ is the probability of choosing Path $k$, given by $p_{k}=\operatorname{probability}\left(C_{k}\right.$ $\leq C_{m}$ ) for all Paths $m$.

The value of $p_{k}$ depends on the distribution of $\varepsilon_{a}$ (link perception). Two distributions commonly used are the Gumbel distribution, which leads to a logit model for path choice, and the normal distribution, which leads to a probit model for path choice. The probit model formulation, although computationally more involved, is preferred. Logit-based route choice modes treat all paths as independent even if they share a large number of common links, whereas probit-based route choice models do not suffer from this limitation. Using the probit model, travel time on Link $a$ is assumed to be normally distributed with mean $t_{a}\left(x_{a}\right)$ and standard deviation $\beta t_{a}\left(x_{a}\right)$, where $\beta$ is the coefficient of variation.

The level of information provided by motorist information systems to the users can be incorporated into the model by different values of the coefficient of variation of the travel time. Given that $t_{a}\left(x_{a}\right)$ is the actual travel time on Link $a$ (for a given flow $x_{a}$ ), then different perceptions of users on that value result in a probability distribution function centered around $t_{a}\left(x_{a}\right)$. Thus, large variance (expressed by a high value of $\beta$ ), corresponds to a situation in which perceptions of travel time differ considerably, whereas small variance indicates good knowledge of travel times (see Figure 1).

When traffic information is provided, the perceptions that users (who receive the information) have of travel times improve and become centered around true travel times. This phenomenon can be expressed by a reduced variance for users with


FIGURE 1 Perceived travel time distribution as a function of information availability.
access to information. The limiting case occurs when perfect information is available and all users know the true value of travel time. In this situation, the perception variance is zero. This analysis assumes symmetry of the probability distribution function both before and after provision of information.

Under this framework, different information systems and information availability can be modeled. In the previous formulation, actual link travel times are not stochastic (they are only perceived differently by the users). Therefore, the models presented cannot be used to address the problem of incident congestion (where the capacity of a link is temporarily reduced and the link travel time increases). Furthermore, incident congestion phenomena are highly dynamic in nature, whereas the traffic assignment model used in this study is static.

## CASE STUDIES

In order to estimate potential benefits of motorist information systems and their sensitivity to various parameters, the model presented previously was applied under different scenarios of information availability and congestion levels on a relatively small suburban network using data from the city of Sudbury, Massachusetts, and a much larger urban network representing the Boston metropolitan area. Although the approach used is aggregate, the trends observed in the case study are indicative of the interactions among the various parameters of the problem and can be used for an initial assessment of the expected benefits from the introduction of motorist information systems.

## Sudbury Case Study

In the town of Sudbury, the road network is characterized by three main routes that stretch from east to west: Routes 20, 27, and 117. The network consists of 204 nodes, 70 centroid nodes, and 578 links of which 214 are connectors joining the centroids to the network. A total of 12,247 trips per hour take place between origin and destination pairs of the network. The standard BPR equation was used for the link performance functions, i.e., travel time as a function of link flows is given by
$t_{a}\left(x_{a}\right)=t_{a}^{0}\left[\left(1+0.15\left(x_{a} / \mathrm{cap}_{a}\right)^{4}\right]\right.$
where $t_{a}^{0}$ is free-flow travel time on Link $a, x_{a}$ is the flow on Link $a$, and $\operatorname{cap}_{a}$ is the capacity of Link $a$.

The practical capacity assigned to the performance function of each link was set at $800,900,1,000$, and 1,100 veh/hr for $9-, 10-, 11-$, and $12-\mathrm{ft}$ lane width, respectively. The free-flow travel time $t_{a}^{0}$ was based on the measured length of the corresponding link and the maximum allowed speed.

In this case study, various scenarios were designed to examine the sensitivity of the expected benefits to the following parameters:

- Level and Amount of Information. Because the amount and level of information are modeled using the coefficient of variation $\beta$, the distribution of flow was compared for different values of $\beta$ that varied from 0.0 (which corresponds to
perfect information and deterministic assignment) to 0.5 (i.e., the standard deviation is equal to 50 percent of the actual travel time). Values of $\beta$ between 0.0 and 0.5 represent intermediate levels of information provided. However, a value of 0.5 is rather high but is used in this study because the objective is to examine sensitivity of benefits to the various parameters of the problem.
- Extent (Spatial Distribution) of Information. Two cases were considered: (a) information is available for virtually all links in the network (e.g., the case of in-vehicle information systems) and (b) information is available only on major routes in the network (e.g., the case of radio broadcasting systems).
- Percentage of Informed Users. Two groups of motorists were used: informed users (i.e., those capable of receiving information) and uninformed users. The percentage of informed users varies from 0 to 100 percent. When a mixed population of motorists is assumed, the perception of uninformed users was modeled by using the highest value of $\beta=0.5$.
- Congestion Levels. For the original origin-destination matrix, the average flow/capacity ratio was about 0.45 . In order to examine the effect of congestion levels, the network was loaded using the original origin-destination matrix multiplied by factors equal to $1.5,2.0$, and 2.5 (resulting in average flow/capacity ratios of $0.67,0.88$, and 1.1 , respectively).

Examination of all combinations of the various cases generated a large number of scenarios.

In order to compare the flow distributions obtained from the various scenarios, travel time per user (informed and uninformed) and total travel time for a given flow distribution were used as measures of performance. However, the data set did not include observed link flows. Consequently, an estimate was difficult to obtain for the value of the coefficient of variation $\beta$ that best describes the current behavior of the users of the system. If data were available for observed link flows, then a value for $\beta$ could be chosen that provides the best fit between link flows predicted by the corresponding SUE and observed link flows. This value of $\beta$ could then be interpreted as reflecting the current level of information availability among the motorists.

Figure 2 presents results for the case where all users have the same perception of travel times. For each value of $\beta$, average path travel time and the corresponding average shortest path travel times are illustrated. A 4.4 percent reduction in average travel time was observed as the coefficient of variation decreases from 0.5 to 0.0 . The time on the shortest path (i.e., the average travel time per user if all users follow the shortest path, at equilibrium, to their destination) increases as $\beta$ decreases because as more information becomes available more motorists make intelligent decisions and follow the shortest path to their destination. However, as more motorists follow the shortest path, travel times (on the shortest path) increase because of the link performance functions used. This behavior of the average shortest path time also indicates that measuring expected benefits of motorist information systems, on the basis of the difference between travel times on the current shortest path (for a given origin-destination pair) and the travel time on alternative paths, overestimates the actual benefits (because this method ignores the interactions among users).


FIGURE 2 Path travel time as a function of information availability.

Table 1 presents the sensitivity of travel times to information for various congestion levels. Three levels of congestion are presented: the base case (load factor 1.0) and cases with load factors 1.5 and 2.0.

The results indicate moderate benefits from the introduction of motorist information systems. The percentage reduction in total travel time between the deterministic case and the case of limited information ( $\beta=0.5$ ) decreases slightly as the congestion level increases. In terms of average travel time per user, the greatest reduction in travel time is experienced when the load factor is 1.5 . The benefits per user decrease as the load factor moves away from the value 1.5 . Table 1 also indicates that additional benefits gained by moving toward a system-optimal allocation of flow are of similar magnitude as the benefits from improved information. Furthermore, these benefits increase with the congestion level (up to some point).

Figure 3 shows the sensitivity of system-wide travel time to the number of users who have access to information, for the base case of congestion levels. The coefficient of variation for uninformed users was fixed at 0.5 , whereas the $\beta$ value for informed users was set at $0.33,0.25$, and 0.0 , respectively. In all cases, travel times decreased linearly with the number of informed motorists. This result is somewhat surprising because the rate of decrease in total travel time is expected to decrease as the percentage of informed users increases. The same trend was observed even when the network was more congested (load factors 1.5 and 2.0). However, when the $\beta$ value of uninformed users becomes 0.45 (which implies that uninformed users have better perceptions than originally), the relationship between travel time and percentage of informed users behaves closer to the expected. Although initially the reduction is linear (but with a much smaller slope than before),


FIGURE 3 Sensitivity of travel time to percentage of informed users when the coefficient of variation for uninformed users is 0.50 .

TABLE 1 SENSITIVITY OF TRAVEL TIME (hr) TO INFORMATION AND CONGESTION LEVELS

| Coefficient of <br> Variability $\beta$ | Congestion Level (x/cap) |  |  |
| :---: | :---: | :---: | :---: |
|  | $1.0(.45)$ | $1.1(.67)$ | $1.2(.88)$ |
| SYSTEM OPTIMAL | 111,810 | 130,871 | 155,614 |
| 0.00 | 114,815 | 136,926 | 159,974 |
| 0.25 | 114,986 | 135,905 | 163,151 |
| 0.33 | 116,957 | 138,172 | 165,807 |
| 0.41 | 119,854 | 140,621 | 168,715 |
| 0.45 | 120,975 | 142,127 | 170,428 |
| 0.50 | 123,545 | 144,703 | 170,642 |

the rate of reduction is lower and the curve tends to be asymptotically horizontal as the percentage of informed users increases. This observation supports the argument previously made that the $\beta$ value of 0.5 used for uninformed users is too high and a lower value may better reflect actual conditions. Unfortunately, because of the lack of data (link observed flows), a proper value for $\beta$ cannot be determined.

Figures 4 and 5 show the differences in travel times between informed and uninformed users. Figure 4 clearly demonstrates that, as expected, the difference between informed and uninformed users increases as more information becomes available to informed users (case with a load factor of 2.0). However, for all scenarios of information availability, the benefits experienced by informed users (over uninformed) decrease as the congestion levels on the network increase. Therefore, the value of information decreases as the network becomes more congested because the opportunities for identifying better paths decrease and both informed and uninformed users experience long travel times. Figure 5 shows the difference in travel times between informed and uninformed users as a function of the number of informed users. Jeffery (1) speculates that "additional sources of potential benefit that would accrue mainly to dynamic guidance systems include: (i) savings in congestion and delays, and hence time and vehicle operating costs, for non-equipped vehicles because equipped vehicles are diverted out of their way . . . ." However, this finding is not supported by the results shown in Figure 5 (at least for the case of recurrent congestion). Overall travel times for uninformed users, in general, increase marginally as the percentage of information-equipped vehicles increases. Similarly, because


FIGURE 4 Comparison of average travel times between informed and uninformed users with $\mathbf{5 0}$ percent informed users when the $\beta$ value for uninformed users is $\mathbf{0 . 5 0}$.


FIGURE 5 Sensitivity of average travel time to percentage of informed users when the $\boldsymbol{\beta}$ values are 0.25 for informed users and 0.50 for uninformed users.
of existing interactions, travel time for informed vehicles also increases. The difference in travel times is substantial enough that overall travel time decreases (see also Figure 3) as the percentage of informed users increases.

Given the probabilistic nature of the traffic assignment, to validate the results further, the hypothesis that link travel times for informed users are statistically less than the corresponding travel times for uninformed users was tested at the 95 percent confidence level. The hypothesis was accepted for all combinations of information availability and congestion levels (with the exception of the case in which the network loading factor was 2.5 and the corresponding average volume/ capacity ratio was greater than 1 ).

Finally, Figures 6 and 7 show the results for the case where information is available only on selected routes in the network. In Figure 6, two cases are shown. In the first case, all users have the same information on all links in the network (see also Figure 2). The second case is identical to the first with the only difference that the information on the three major routes (Routes 20, 27, and 117) is perfect ( $\beta=0$ ). As expected, the difference in travel time between the two cases decreases as more information is available to the users. For all values of $\beta$, the benefits from the additional information on the three main routes are small. Figure 7 shows the differences in travel times for two cases: in the first case all users have information on the three major routes and no information ( $\beta=0.5$ ) on all other routes, whereas in the second case motorists have information on all routes. The level of information varies from $\beta=0.5$ to $\beta=0.0$. The results clearly demonstrate that the value of spatial information increases as


FIGURE 6 Effects of the provision of information on selected routes.


FIGURE 7 Average travel time as a function of spatial availability of information.
the level of information increases. When $\beta=0.0$ (i.e., perfect information on the three routes in the first case, and perfect information on all links for the second case), travel time decreases by 4 percent as the spatial availability of information extends to all links.

## Boston Case Study

The second case study was performed on a larger urban network describing the Boston metropolitan area. The network consists of 7,724 nodes; 239 centroid nodes; and 11,647 links, of which 1,064 are connectors with approximately 186,578 trips per hour. The Central Transportation Planning Staff (CTPS) in Boston provided the data used in this case study.
The BPR equation was used for link performance functions with a power of 6 for the flow/capacity ratio. The same scenarios as the Sudbury data were examined except extent of information, which has not yet been tested because of the size of the network and the existence of many major routes, some of which have nontrivial overlap. Because the Boston network is more congested than the Sudbury network, lower load factors of 1.1 and 1.2 were applied. Generally, the same trends were observed, but in some cases more insight was provided and some counter-intuitive phenomena that appeared in the first case study were addressed.
In Figure 8, in which all users have the same perception of travel time, the same trends as in Figure 2 appear. However, the magnitudes are quite different. In the Sudbury data, average travel time reduction was 4.4 percent as $\beta$ decreased from 0.5 to 0.0 , whereas in the Boston data this decrease amounted


FIGURE 8 Path travel time as a function of information availability.
to 8.4 percent. On the other hand, average travel time along the shortest path increased by 0.8 percent in the Sudbury case study and by 3.2 percent in the Boston case study as $\beta$ decreased. This result could be explained by the fact that the shortest path becomes congested faster because of the higher power of the flow/capacity ratio in the BPR function and the higher level of congestion in the network. The differences reported are for the two extreme values of the coefficient of variation. Unfortunatcly, the lack of obscrved link flows again does not allow the determination of a value for $\beta$ that best represents existing conditions. However, a priori the true value is expected to be lower than 0.5 . Furthermore, the average travel time reported does not include the time to traverse the connectors from or to the origin-destination centroids.

An interesting result is presented in Table 2 that compares total travel times in the network among the various scenarios of information availability and load factors. The total travel time under system optimal conditions is also given. Surprisingly, traffic conditions for the case of load factor equal to 1.1 (measured by total travel time) worsen when full information is provided compared with limited information availability ( $\beta=0.25$ ). The result, which suggests that information may not always improve travel times, needs further investigation because the difference observed is possibly caused by convergence effects of the model used rather than behavioral reasons (although in all other cases a monotone reduction in tràvel times is observed as more information is provided).

Figure 9 shows a more intuitive relation between total travel time and the percentage of informed users than Figure 3. For each of the three cases examined, a decreasing S-shaped function demonstrates that when a small percentage of users has access to information, total decrease in travel time is moderate. As more users gain access to information, travel time decreases at faster rates (when the percentage of informed users is somewhere between 30 and 70 percent). Finally, a small marginal decrease in travel time occurs when the percentage of informed users increases above a certain value. This pattern agrees more with the prior expectation than the linear rate of decrease that appeared in Figure 3 and supports the hypothesis that marginal benefits from providing information decrease as more users gain access to the same kind of information.


FIGURE 9 Sensitivity of travel time to percentage of informed users with a $\beta$ value for uninformed users of $\mathbf{0 . 5 0}$.

Figure 10 shows additional information on the behavior of average travel time experienced by uninformed users as larger numbers of motorists have access to and use traffic information. Although in Figure 5 a general trend of increased travel time for uninformed users was observed, Figure 10 clearly shows nonmonotone behavior of the average travel time experienced by uninformed users. Travel time is lower when approximately half of the users in the system are informed. These results support the hypothesis that uninformed users may also benefit from the fact that informed users make better decisions, some of which involve route diversions. However, as the percentage of informed users increases beyond approx-

TABLE 2 SENSITIVITY OF TRAVEL TIME (hr) TO INFORMATION AND CONGESTION LEVELS

| Coefficient of <br> Variability $\beta$ | Congestion Level (x/cap) |  |  |
| :---: | :---: | :---: | :---: |
|  | $1.0(.45)$ | $1.5(.67)$ | $2.0(.88)$ |
| SYSTEM OPTIMAL | 1,249 | 2,082 | 3,352 |
| 0.00 | 1,260 | 2,139 | 3,503 |
| 0.25 | 1,270 | 2,155 | 3,523 |
| 0.33 | 1,283 | 2,169 | 3,546 |
| 0.41 | 1,297 | 2,191 | 3,585 |
| 0.45 | 1,305 | 2,204 | 3,608 |
| 0.50 | 1,318 | 2,222 | 3,637 |



FIGURE 10 Sensitivity of average travel time to percentage of informed users when the $\boldsymbol{\beta}$ values are $\mathbf{0 . 2 5}$ for informed users and $\mathbf{0 . 5 0}$ for uninformed users.
imately 70 percent, informed and uninformed users start to experience an increase in their respective average travel times.

Finally, the results from the Boston case study should be interpreted with caution for various reasons: (a) analysis of the data is not yet complete, and (b) data obtained from CTPS was not in its original form but had already been calibrated to some extent.

## CONCLUSION

A methodology to analyze long-term effectiveness of motorist information systems in reducing recurrent congestion was described. The methodology is based on the assumption that provision of information affects the perception users have on link travel times of a network and therefore improves their route choice. The model was applied on two networks (urban and suburban) and provided some understanding of the interactions among the most important parameters of the problem such as percentage of informed users and level and extent (spatial coverage) of information. The results show that motorists experience modest reduction in travel times because of the introduction of motorist information systems. The gains, though, are proportionally smaller than the ones estimated by other studies. However, these differences are attributable to the fact that previous studies have ignored interactions among users of the system.

From a theoretical point of view, the methodology presented has several limitations, and current research concen-
trates on relaxing the assumptions underlying the models used. Some of the most important of these limitations are as follows:

1. No formal link established by the research so far exists between the level and extent of information and the corresponding value of $\beta$. Such a link, if established, could be useful in measuring the value of information more accurately. Furthermore, this relationship could be used to evaluate the most appropriate hardware configuration.
2. The approach presented is only useful in assessing longterm benefits caused by reduced recurrent congestion from a motorist information system. Further work is necessary to capture similar effects under incident congestion conditions.
3. The models presented cannot be used directly to assess the benefits from using the information as a control variable to obtain flow distribution closer to the system-optimal distribution, in which use of the network is maximum, a goal that may be neither realistic nor feasible, anyway.
4. The model used is static and does not allow the study of other important effects, such as changes in departure times.

The resolution of these limitations-especially for evaluating benefits caused by a reduction in incident congestionrequires better understanding of user perceptions for route choice modeling. From a practical point of view, a need exists for conducting case studies using better modeling of link travel time perception and effects of information and examining the sensitivity of the results to other important parameters such as coordination of motorist information systems and traffic signals in a way similar to the Long Island study previously mentioned (8).

In summary, the modeling framework, despite limitations, has provided a means to incorporate information availability and its effect on traffic congestion, to identify important interactions among the important components of the problem, and to evaluate potential long-term benefits.

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# Motorist Behavior and the Design of Motorist Information Systems 

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#### Abstract

Results are described of an in-person survey of Seattle commuters on the topic of motorist behavior and decision making. Three broad areas were of interest: (a) behavior and decisions of commuters relative to their choice of route before departure; (b) behavior and decisions of commuters while driving; and (c) responses of commuters to manipulations of variable-message sign messages. Results indicated that all commuter groups were familiar with alternative routes, but rarely used them during actual commutes. In addition, commuters who did use alternate routes reported higher levels of stress during their commutes in comparison with traveling primary routes. Changes in primary routes were generally in response to congestion actually observed by commuters rather than to information provided by existing traffic information sources. Further, commuters rarely changed mode of transportation or time of departure on the basis of currently available information received at home before departure. These findings point to specific issues that need to be addressed in the design of future motorist information systems.


As politicians, planners, and engineers in major metropolitan areas have struggled to meet the transportation needs of their citizens, emphasis has been placed on surveying the commuting population to increase the available knowledge of general driving characteristics (1-7). These surveys have tended to focus on demographic features of the population, such as age, sex, and income. The critical problem with this general approach is that often the assumption is made that the commuting population can be treated as a homogeneous audience of motorists with respect to driving behaviors, decision processes, and needs for traffic information.

In fact, motorists cannot be treated as a homogeneous population. As demonstrated by more recent approaches to the analysis of commuting behavior $(8,9)$, including studies of motorist behavior in the metropolitan Seattle area $(10,11)$, stable subgroups of the commuting population can be identified and the identification of these subgroups can be used (among other purposes) to optimize the means of delivering traffic information to members of these subgroups. By basing the design of motorist information systems on knowledge of the behavior of identifiable subgroups of commuters, traffic planners can deliver user-based traffic information with the goal of allowing drivers to make decisions that will result in fewer delays for individual motorists and an overall improvement for commuters in general (12).

[^23]Procedures and results of an in-person interview of Seattle commuters on motorist behavior and decision making are described. The survey used a random sample of commuters ( $n=96$ ) drawn from a larger sample of motorists ( $n=1,697$ ) who participated in an earlier and more general study of motorist behavior and information needs.

## BACKGROUND

In September 1988, investigators from the University of Washington's College of Engineering, in cooperation with traffic engineers from the Washington State Department of Transportation (WSDOT), conducted a major study of the commuting population in metropolitan Seattle. The goal was to provide a set of recommendations aimed at improving the design and delivery of traffic information, thereby increasing the likelihood of changing commuters' driving behavior and improving the flow of peak-time traffic. Nearly 4,000 drivers who commute into downtown Seattle on southbound I-5 completed mail-in surveys, allowing the investigators to gather results on 62 variables relevant to motorist behavior and the impact of available motorist information on commuters' route decisions. Motorists commuting from areas north of downtown Seattle were selected for the sample frame because they have access to a range of motorist information sources, including commercial radio, highway advisory radio (HAR), and variable-message signs (VMS).

Using a multivariate statistical procedure termed cluster analysis, the investigators identified four commuter subgroups (clusters) based on commuters' willingness to adjust their driving behavior in response to traffic information (13). The four clusters were defined as follows:

1. Route changers ( 20.6 percent), those willing to change their commuting route on or before entering I-5;
2. Nonchangers ( 23.4 percent), those unwilling to change departure time, transportation mode, or commuting route;
3. Time and route changers ( 40.1 percent), those willing to change both departure time and commuting route; and
4. Pretrip changers ( 15.9 percent), those willing to make time, mode, or route changes before leaving the house but unwilling to change en route.

Route changers often divert to an alternative route on I-5 and report that' traffic information often influences their route choice but not their departure time or transportation mode. Nonchangers rarely divert to alternative I-5 routes and rarely
or never change their departure time, transportation mode, or commuting route. Time and route changers sometimes divert to an alternative I-5 route, often change their departure time and commuting route, but rarely change their transportation mode. Pretrip changers often alter their departure time and sometimes change their transportation mode, but rarely divert to alternative routes once on I-5.

Whereas the cluster analysis was conducted to identify groups of commuters based on their willingness to adjust their commute in response to traffic information, further analysis was conducted to determine commonalities among commuter responses in the survey. A principal components factor analysis of the correlation matrix from the original data set revealed a five-factor solution across the 62 variables (11):

1. Issues affecting route choice;
2. Distance-time information;
3. Traffic information, particularly TV and radio;
4. Traffic information, particularly VMS, HAR, and telephone hot line; and
5. Commute attributes and flexibility.

This initial survey provided an extensive amount of data on commuter behavior. The survey revealed that the commuting population of metropolitan Seattle is diverse and cannot be treated as a homogeneous audience for motorist information, yet it is composed of stable subgroups. However, this initial survey also raised a number of interesting questions, which the in-depth interview was designed specifically to probe. These questions assessed flexibility in departure and arrival time, commuters' specific knowledge of primary and alternative routes, and detailed characteristics of the commute. From the original survey, cluster analysis provided a means of conceptualizing the design of the in-depth survey and principal components factor analysis indicated that additional information was required regarding issues that might affect route choice.

## METHOD

## Subjects

Subjects were recruited from a portion of the initial survey respondents who indicated a willingness to participate in an in-depth study. Of the 3,893 respondents in the initial survey,

1,697 agreed to the in-depth study. Subjects in the original survey entered I-5 on one of the seven downtown exits.

In order to be included in the original sample frame, motorists had to (a) travel south on some portion of the north I-5 corridor to downtown Seattle during peak morning commute hours (6:30 to 9:00 a.m.), (b) travel this corridor at least once each week, and (c) be the driver of the commuting vehicle. This group of commuters allowed focusing on drivers exposed to a variety of highway information sources and drivers with a variety of end points for their commutes.
Subjects for the in-depth survey were selected at random from each of the four clusters identified in the initial survey, in numbers proportional to the original cluster sizes. Table 1 presents the distribution of subjects according to sex and cluster membership. Of 120 subjects who were recruited, 96 subjects participated, an 80 percent participation rate.

## Variables of Interest

The in-depth interview probed three broad areas of interest, on the basis of the analysis of responses to the initial survey:

1. Behavior and decisions of commuters relative to their route choice before departure,
2. Beinavior and decisions of commuters while driving, and
3. Responses of commuters to a set of syntactic and semantic manipulations of messages that might be displayed on VMS.

The survey was limited in temporal scope to the time of the inbound commute. This decision was partially based on the initial observation of significant differences in commuters' flexibility in departure on the morning commute with minor differences observed for the afternoon commute. Also, because the literature on survey methods indicates that the total inperson interview time should be less than 1 hr for all aspects of an interview [e.g., see work by Sharp and Frankel (14)], probing both inbound and outbound commutes was deemed infeasible.

## Behavior and Decisions Before Departure

The first set of questions investigated the behavior of and decisions made by commuters in the period before departing for work. In order to understand the environment in which

TABLE 1 SAMPLE DISTRIBUTION ACCORDING TO SEX AND CLUSTER

| Cluster | Males | Females | Total | $\%$ of Total |
| :--- | :---: | :---: | ---: | ---: |
| Route changers | 14 | 12 | 26 | $21.9(20.6)$ |
| Non-changers | 10 | 12 | 22 | $22.9(23.4)$ |
| Time and route changers | 12 | 1.5 | 27 | $28.1(40.1)$ |
| Pre-trip changers | 11 | 10 | 21 | $21.9(15.9)$ |
|  | 47 | 49 | 96 |  |
|  | $49 \%(49 \%)$ | $51 \%(51 \%)$ |  |  |

Note: Numbers in italics indicate percentages each cluster represented in the original survey.
pretrip information is received and used, the amount of time that commuters have before departure was examined along with the number and type of demands experienced by commuters during that time. Commuters were asked to estimate the time they normally awaken and normally leave for work. These estimates were used to calculate the average amount of time that commuters have before departure. Commuters were also asked to estimate the number of significant tasks completed during this period. Significant tasks were defined as any tasks outside of those completed in preparation for departure. Commuters were also asked to give subjective estimates of the amount of stress experienced during the period before departure and the amount of flexibility in determining departure time.

Also included in this first set were questions probing access to and use of traffic information during the period before departure. Commuters were asked to describe how actively they seek traffic information, how frequently they receive traffic information, and how they make decisions on the basis of that information. Commuters also estimated the amount of time between first receiving information and departing. Commuters were then asked about their decisions to alter choice of route, mode, and departure time.

## Behavior and Decisions En Route and After Commute

The second set of questions probed commuters' behavior and decisions during the commute between home and work. Commuters were asked to orally describe one primary and two alternative routes (if known and used) and to trace those routes on detailed street maps. Commuters' knowledge of the routes was assessed by counting the number of landmarks, street names, and compass directions used in their descriptions of primary and alternative routes, a method suggested from human factors investigations of route knowledge, navigational skills, and map reading $(15,16)$.

Tallies were made of the total number of decision points on primary and alternative routes. Decision points were defined as points where drivers might need to make adjustments to their routes. Commuters were asked to report their reasons for altering routes at each of the decision points and their use of traffic information (including their subjective observations of traffic conditions) in making choices. Additional questions probed sources of information used to either confirm or refute decisions to alter routes. The final items in this set of questions probed commuters' perceptions of flexibility in arrival time, penalties for arriving late, and stress of using alternative routes. Commuters were also asked to estimate the number of times they arrived late for work each month because of traffic conditions.

## Syntactic and Semantic Manipulations of Sign Content

The third set of questions probed commuters' responses to syntactic and semantic manipulations of messages that might be displayed on VMS. Two messages that WSDOT typically displays on a VMS located above southbound I-5 just north of downtown Seattle were used as the basis for the two sets of manipulations.

The first set of manipulations involved two variables known to affect task performance - task instruction and message order (17-19). Two types of task instruction were presented-specific and generic. Messages that presented specific task instructions suggested a specific alternative route in response to a traffic situation (e.g., Use I-90 Eastbound); messages that presented generic task instructions suggested a general response (e.g., Use Alternate Routes). The order of messages was randomized so that the task instruction (either generic or specific) appeared either before or after the reason (the description of the traffic problem).
The second set of manipulations involved the type of reason presented in the message and the presence or absence of the task (the response to the traffic situation). Two types of reasons were presented, specific and generic. Messages that contained specific reasons presented a specific description of the traffic problem (e.g., Accident at Mercer Street Exit); messages that presented generic reasons gave a more general description (e.g., Accident Ahead). Messages were manipulated to present either a suggested response to the traffic situation or a general statement (e.g., Expect Delays).

Messages were printed in 24-point Helvetica bold on $81 / 2$ $\times 11-\mathrm{in}$. white paper in landscape orientation using a laser printer. Commuters' interpretation of sign content and indication of a probable response to the message were used as dependent measures rather than recall, following observations by human factors researchers indicating superiority oí reaction behaviors to recall in assessing significance of sign content $(20,21)$.

## Survey Administration

The choice of survey administration method considered a number of issues: demands placed on the subjects, reinforcement of subjects, reliability, response mode, and tone and presentation. The in-person format was selected on the basis of a review of the literature and after considering the issues surrounding the types of questions to be asked. A number of authors have suggested that as demands placed on the respondent are reduced, the quality of responses increases $(14,22)$. Babbie (22) notes that not only do in-person interviews have higher response rates than other survey methods, but that in-person interviews also allow the interviewer to clarify questions when the respondent is confused. Additionally, inperson interviews allow a wider set of responses to items in the survey.

In-person interviews allow subjects to be positively reinforced for their participation, both at the outset and at the conclusion of the interview, thus increasing subjects' sense of the importance of their contribution to the survey. Sharp and Frankel (14) note that if subjects perceive their contributions to be important, responses will be of higher quality.

Although in-person interviews have numerous advantages, threats to internal validity must be controlled. The nature of in-person interviews can create a lack of reliability in response sets caused by inconsistencies among interviewers and among interview sessions conducted by individual interviewers. Two steps were taken to control for these two threats to validity. First, interviewers were trained by one researcher and were required to meet specific criteria before interviewing com-
muters. Second, interviewers worked from a written questionnaire that specified all interviewer prompts and provided categories for recording commuter responses.
The formal training of interviewers occurred in two parts. The first part of the training consisted of the trainer reading each of the questions aloud, as they should be read to commuters, and discussing the possible responses. During this part of the training, the trainer also discussed timing and preparation for the interview. The second part of the training required the trainee to interview the trainer. The trainer was prepared with a set of difficult or ambiguous responses to a number of the interview questions. The trainee was required to respond to and code the responses correctly two times without error before being considered trained on a specific question. Each interviewer received approximately 3 hr of direct training before interviewing commuters. In addition, each interviewer was required to have partially memorized the scripted portions of the interview before the formal training. Once trained, interviewers were debriefed by the trainer following randomly selected interviews to determine if any retraining was required.
The tone of the interview was intended to be conversational but neutral. Interviewers were to present themselves as interested professionals to enhance commuters' sense of the importance of their responses without encouraging specific response patterns.
Subjects were interviewed individually at the University of Washington. They were informed that participation was voluntary and that they could take a break or terminate the interview at any time.

## Data Coding and Analysis

All categorical variables were coded for data entry following completion of the entire set of interviews. A standardized coding protocol was established in advance, assigning numeric codes to the categories of responses in each of the interview questions. Coding of all interviews was completed by two research assistants. Coding accuracy was subsequently checked and all errors were corrected before data entry. All statistical analyses were conducted using the SYSTAT statistical software system.

## RESULTS AND DISCUSSION

Data obtained in the in-depth interview were analyzed in a manner similar to the data analysis completed for the first survey. Responses were examined for patterns across the entire sample and were further examined for patterns across clusters and sex. Finally, a principal components factor analysis was conducted to reveal commonalities of responses.

## Patterns Across the Entire Sample

## Behavior and Decisions Before Departure

Commuters reported that they have approximately 72 min ( $\mathrm{M}=71.969, \mathrm{SD}=32.391$ ) between the time they wake up and the time they depart for work. During that period, commuters must accomplish at least one significant task ( $\mathrm{M}=$
$1.063, \mathrm{SD}=1.296$ ) other than preparing themselves to leave, such as preparing breakfast for other members of the household. The majority of commuters reported that they perceive this period to be relatively calm ( 60.42 percent rated the period as a 1 or 2 on a 5 -point scale of hecticness) and relatively stress free ( 66.67 percent rated the period as a 1 or 2 on a 5 -point scale of stress).
The majority of commuters ( 72.92 percent) receive traffic information of some kind during the period before departure and reported that traffic information is first received soon after awakening. Half of the commuters reported receiving traffic information pertaining to their primary route almost immediately after awakening. An additional 10 percent (for a total of 61.43 percent) reported that they receive their first traffic information more than 1 hr before departure. Commuters reported receiving traffic information at least three times $(\mathrm{M}=3.059, \mathrm{SD}=2.143)$ between awakening and departing. Although commuters reported that they are aware of traffic information, they reported that the information has little impact on their decisions before departure. The majority of commuters reported that they rarely decide to use an alternate route ( 65.71 percent), that they rarely decide to use an alternate mode ( 90.00 percent), and that they rarely decide to change their departure time ( 64.29 percent) on the basis of information received before departure. In an average month, commuters reported that before departure they decide to change their route twice ( $M=2.333, S D=2.666$ ). Although these results might indicate that a large number of Seattle commuters are unresponsive to traffic information delivered before departure, the results also indicate that an important proportion of commuters could be influenced by predeparture traffic information. Indeed, if traffic information could influence one-third of Seattle commuters to change their departure time or route choice or one-tenth to change transportation mode, significant improvements in peak-time traffic conditions could result.

On the whole, commuters are somewhat receptive to traffic information delivered before departure. Commuters reported that the period before departure is not stressful and that they have a relatively small number of tasks to accomplish. The low rate of modification to route, mode, and departure time may indicate that while receiving traffic information, commuters may receive the information passively and may not find it credible. This second inference is supported by comments to this effect received from commuters on the initial survey (10). The low rate of route modification may as well be caused by temporal delay between receipt of the information and decision because the majority of commuters reported receiving their first traffic information more than 1 hr before departure.
For purposes of designing an information system, these rcsults reinforce the notion that demonstrating system credibility may be a significant issue. Further, these results indicate that commuters may have time to use an interactive graphical traffic information system, one that would demand some active engagement.

## Behavior and Decisions En Route

Commuters indicated a high degree of knowledge about their primary commuting route and their first and second alter-
native routes. Alternative routes were defined as major deviations from the primary route that could, however, include a small portion of the primary route. Thus, an alternative route could include a portion of I-5. Indications of route knowledge were obtained from counts of the number of landmarks and street names used when commuters described their commuting routes. Results support the intuitive prediction that commuters have a more detailed knowledge of their primary route than of either their first or second alternative routes. For example, when asked to trace their commuting routes, commuters used five times as many street names as they did landmarks in describing their routes. Table 2 presents the means and standard deviations for number of street names and landmarks used by commuters to describe their primary route and their first and second alternatives.

Of commuters who both know and use an alternative route, half reported that their first and second alternative routes avoid I-5 (52.94 percent for the first alternative route, 50.90 percent for the second alternative route). The vast majority of commuters ( 95.83 percent) reported knowing an alternative to the route that would be used if a large portion of their normal route were inaccessible for some reason. On average, commuters reported knowing between two and three alternative routes ( $\mathrm{M}=2.880, \mathrm{SD}=1.568$ ). However, only 75.00 percent of those interviewed reported that they actually use one of those alternatives.

Commuters reported that the decision to use an alternative route is based first on traffic information received in the car ( 33.28 percent for the first alternative route, 35.09 percent for the second alternative route) and second, on observed traffic conditions ( 23.53 percent for the first alternative route and 21.05 percent for the second alternative route). Interestingly, approximately one-fourth of the commuters who use alternative routes reported that they seek out information about the use of an alternative route while at home, more than 30 min before departing ( 26.87 percent for the first alternative route, 24.56 percent for the second alternative route).

Commuters reported receiving little feedback regarding their choice to use an alternative route and what feedback they do receive is relatively delayed. Nearly one-third of the commuters indicated that they have no way of telling if their choice to use an alternative route is correct or not (27.94 percent for the first alternative route, 31.58 percent for the second alternative route). The majority of commuters indicated that if they do receive any kind of information confirming or refuting their choice to use an alternative route, they receive this information more than 5 min after making the choice ( 69.57 percent for the first alternative route, 48.72 percent for the second alternative route). Only a small percentage of commuters
(2.94 percent) indicated that they receive this information from radio traffic reports.

Commuters reported making between one and two adjustments to their normal route each day ( $\mathrm{M}=1.552$, $\mathrm{SD}=$ 1.897). Adjustments were defined as minor deviations from the primary route that necessarily include a return to the primary route. Primarily, route adjustments are made in response to observed traffic congestion and reports of traffic congestion received in the car (i.e., radio traffic reports). At the first decision point, observed traffic congestion was cited by 39.47 percent of commuters as the reason for adjusting routes, whereas traffic information was cited by 35.71 percent of commuters. At the second decision point, the percentage of commuters who respond to observed traffic conditions as opposed to traffic reports is even greater: 54.76 percent cited observed traffic conditions as the reason for making their second route adjustment, whereas 23.81 percent cited traffic information received in their cars. If they commit to using an alternative route, commuters reported making fewer adjustments to their alternative routes than they do to their primary route (for the first alternative route, $\mathrm{M}=0.647, \mathrm{SD}=1.182$; for the second alternative route, $\mathrm{M}=0.474, \mathrm{SD}=0.928$ ).

On the whole, commuters reported having few tasks ( $\mathrm{M}=$ $0.611, \mathrm{SD}=0.982$ ) to complete (such as dropping off a family member) on their normal drive into the city. Although the majority of commuters ( 58.95 percent) reported experiencing low to moderate levels of stress on their primary route, approximately one-third of the commuters ( 33.68 percent) reported experiencing relatively large amounts of stress on their primary route. If they decide to use an alternative route, 77.78 percent of commuters reported that the level of stress experienced increases. Thus, making the choice to use an alternative route (and, perhaps, the driving conditions that lead to that choice) appears to be perceived as a stressful event.

The patterns observed for all commuters indicate that commuters have a high degree of knowledge of their primary and alternative routes, that a majority do at some time make use of alternative routes, and that nearly half of the alternative routes make use of some portion of I-5 (the primary route used by commuters into downtown). Commuters appear to make a small number of adjustments to their primary route, mainly on the basis of their observations of traffic conditions. However, commuters appear to decide to use an alternative route on the basis of traffic reports received either at home or in the car. Commuters receive little feedback regarding their choice to use an alternative route and, when received, this feedback is delayed. Finally, commuters are not burdened with tasks other than commuting to their workplace, and

TABLE 2 NUMBER OF NAMES AND LANDMARKS IN ROUTE DESCRIPTIONS

|  |  | Primary Route | Alternale 1 | Alternate 2 |
| ---: | ---: | ---: | ---: | ---: |
| Strect names | Mcan | 8.45 | 5.02 | 4.26 |
|  | SD | 6.23 | 3.70 | 4.01 |
| Landmarks | Mean | 1.67 | 1.03 | 0.79 |
|  | $S D$ | 1.89 | 1.48 | 0.90 |

approximately one-third of all commuters experience high levels of stress, with the perceived level of stress increasing if they use an alternative route.
The implications for design of an information system for commuters en route are somewhat similar to those for an information system designed for commuters before departure. Although commuters do rely on traffic reports, they require that the information be more current and more specific than the information currently available and that the accuracy of information be verified through feedback. These findings parallel results and recommendations reported nearly two decades ago (23). Increasing currency and specificity might well increase the probability that commuters would choose an alternative route (as opposed to merely making minor adjustments to the primary route). Incorporating feedback mechanisms into on-road systems for delivering information might also increase their effectiveness in encouraging commuters to choose alternative routes.

## Behavior After Commute

Commuters reported that they are late for work because of traffic conditions approximately four times in an average month ( $\mathrm{M}=4.097, \mathrm{SD}=4.099$ ). When asked to rate their flexibility in arrival times, commuters responded with answers that were distributed evenly across a 5 -point scale, indicating no particular pattern. The majority of commuters ( 82.29 percent) indicated that the penalties for arriving late for work are relatively minor (rating the penalties as either a 1 or 2 on a 5 -point scale).

## Summary of Implications for the Design of Information Systems

This description of the behavior and decisions of commuters before departure, en route, and after the commute has a number of implications for the design of information systems:

1. Commuters may benefit from two different types of information systems, one used predeparture and one used en route;
2. These two systems should be integrated to provide feedback and confirmation of accuracy; and
3. The information transmitted needs to be current and specific to be used and acted on.

However, these implications are somewhat limited in scope and do not address the need to change commuter behavior under specific conditions. The analyses reported in the following sections (patterns across commuter group and sex and the patterns observed in the factor analysis) provide a more detailed view of commuter responses and underscore the idea that commuters are not a homogeneous population with uniform traffic information needs.

## Patterns Across Clusters

Group membership for each commuter participating in the follow-up survey was determined on the basis of cluster analysis performed on the data from the initial survey. The four clusters were defined as (a) route changers (RC), (b) nonchangers (NC), (c) time and route changers (TRC), and (d) prechangers (PC). Responses of the commuters participating in the follow-up survey were examined to determine if there were any significant differences in behavior and decisions attributable to cluster membership. This section reports those analyses that produced results significant at $p \leq 0.05$; results with probability values $>0.05$ but $\leq 0.10$ are reported as trends.
A Kruskal-Wallis test for group differences revealed that clusters differed with respect to ilexibility in departure time. Table 3 presents the Kruskal-Wallis statistics, rank values, and probability values for the comparisons reported. As Table 3 indicates, clusters differed significantly in flexibility with regard to departure time - the TRC cluster had the highest

TABLE 3 KRUSKAL-WALLIS COMPARISONS

|  | Rank Value |  |  |  |
| :---: | :---: | :---: | :---: | :---: |
|  | RC | NC | TRC | PC |
| Flexibility in departure time, $H=10.846, p \leq$ | 1328.0 | $6,61.5$ | 1.560 .5 | 1106.0 |
| 0.013 |  |  |  |  |
| Likelihood of changing route due to information | 700.5 | 2.52 .10 | 8666.0 | 666.5 |
| received prior to departure, $H=10.337, p \leq$ |  |  |  |  |
| 0.016 |  |  |  |  |
| Knowledge of alternate routes, $H-7.376, p \leq$ | 1313.0 | 916.5 | 1366.0 | 1060.0 |
| 0.061 , trend |  |  |  |  |
| Activcly seck information on primary routc, $/ l=$ | 876.5 | 477.0 | 617.0 | 514.5 |
| $6.863, p \leq 0.076$, trend |  |  |  |  |
| Scek information prior to departure, $H=9.757$, | 493.0 | 209.0 | $6,32.0$ | 319.0 |
| $p \leq 0.021$ |  |  |  |  |
| Stress on primary routc, $H=7.650, p \leq 0.054$ | 1143.0 | ()12.0 | 1211.0 | 1304.0 |

flexibility, followed by the RC, PC, and NC clusters. Members of the TRC cluster are also significantly more likely to change their route on the basis of information received before departure. The TRC cluster was followed, in terms of decreasing probability of changing route on the basis of information received before departure, by the RC, PC, and NC clusters, a pattern similar to the response to the departure flexibility question.

A one-way analysis of variance (ANOVA) revealed differences between clusters in terms of the number of times cluster members choose an alternative route in an average month, $F(3,74)=4.111, p \leq 0.009$. A Tukey HSD comparison revealed that although there was no difference between members of the TRC and PC clusters (TRC M $=3.29$, PC $M=3.40$ ), members of both clusters select an alternative route more frequently in an average month than members of the NC cluster ( $\mathrm{M}=1.10$ ), (TRC versus $\mathrm{NC}, p \leq 0.0319$; PC versus NC, $p \leq 0.0318$ ). Finally, a Kruskal-Wallis test for group differences indicated that members of the NC and PC clusters tended to have less route knowledge than members of the RC and TRC clusters ( $p \leq 0.061$ ).

The Kruskal-Wallis test revealed a trend ( $p \leq 0.076$ ) for members of the RC cluster to more actively seek out information regarding traffic conditions on their primary route. The RC cluster was followed by the TRC, PC, and NC clusters. However, members of the TRC cluster seek out information regarding traffic conditions more frequently before departure than (in order) members of the RC, PC, and NC clusters.

An ANOVA analyzing the number of landmarks used to describe the first alternative route revealed significant differences among the clusters, $F(3,64)=3.413, p \leq 0.023$. A Tukey HSD comparison revealed that members of the NC cluster used more landmarks ( $\mathrm{M}=2.083$ ) in describing their primary and first alternative routes than members of the other clusters ( $\mathrm{RCM}=1.143, \mathrm{PCM}=0.6471, \mathrm{TRC} \mathrm{M}=0.5556$ ). A heavier use of landmarks as opposed to street names indicates that NC cluster members possess a less detailed knowledge of the route $(15,16)$. A trend was also observed for members of the NC cluster to use more landmarks (as opposed to street names) in their descriptions of their primary routes, $F(3,92)=2.308, p \leq 0.082$.

Although members of the NC cluster appear to have less knowledge of their primary and first alternative routes, they also appear to experience less stress when using their primary route ( $H=7.650, p \leq 0.054$ ). The NC cluster was followed (in terms of increasing stress on the primary route) by the RC, TRC, and PC clusters.
These findings more fully complete the picture of the commuter groups identified in the initial survey and tell more about targating information for these groups. For example, data from the first survey indicate that NC cluster members found traffic information received at home less preferable and had less positive reactions to messages and media. Results from the in-depth survey indicate that members of the NC cluster are more likely to use landmarks than street names in describing their commuting routes, indicating less knowledge of these routes. Because the majority of available traffic information sources rely heavily on the use of street names in the description of routes, members of this cluster would be less likely to find the information usable. Thus, an information system targeting members of the NC cluster might need to
provide more graphic information, including real-time displays of traffic situations, such as live video displays of traffic conditions. The information system might also need to provide greater levels of information regarding alternate routes (perhaps even offering an option that would increase commuters' familiarity with the available routes, in the fashion of a tutorial).

## Patterns Across Sex

Only a small number of sex differences were uncovered in the analyses of the responses to the in-depth interview. Women indicated that they have less flexibility in the time of arrival $\left(\chi^{2}(5)=12.599, p \leq 0.014\right)$ and that they tend to have less flexibility in the time of departure $\left(\chi^{2}(4)=8.700, p \leq 0.069\right)$. Women also tended to rate the period before departure as more hectic than did men $\left(\chi^{2}(4)=9.187, p \leq 0.057\right)$.
These findings imply that women, having greater time demands placed on them, need an information system that provides time estimates of traffic delays and commuting routes. In a more general sense, these findings indicate that commuters differ even by sex with regard to aspects of their commute and use of traffic information, thus supporting further the notion that commuters cannot be treated as a single group in terms of traffic information needs.

## Results of the Factor Analysis

In the previous discussions, patterns were presented that could be attributed to characteristics of commuters (such as sex) and their commuting tasks (such as miles traveled). As was done for the initial survey, a principal components factor analysis was performed on the responses to the in-depth interview to determine commonalities of responses rather than distinguishing member characteristics. In essence, although cluster membership, sex, and distance traveled allowed commuters to be distinguished, principal components analysis allowed the responses of commuters to be analyzed for common features. The purpose of principal components analysis was to elicit the basic structure of the correlation matrix (24). Each resulting factor represents attributes that are highly intercorrelated but not correlated with other attributes.
The five-factor solution obtained (see Table 4) has an interesting degree of conceptual overlap with the five factors obtained for the initial survey (issues affecting route choice, distancetime information, traffic information-TV and radio, traffic information-VMS, HAR, phone, and commuter attributes and flexibility). Because the absolute value rather than the signed value of the loading is of importance, Table 4 presents all loadings with positive values. The matrix of factor loadings was obtained using the VARIMAX rotation; the five-factor solution accounted for 71.82 percent of the total variance in the correlation matrix.

Just as knowledge of the commonalities of the responses of commuters participating in the first survey allowed refinement of the in-depth survey, knowledge of the commonalities of responses in the in-depth interview allows an even finer set of conclusions to be reached regarding traffic information systems. Further, these factors reinforce the importance of not considering commuters as a homogeneous group when designing motorist information systems.

TABLE 4 FACTOR LOADINGS FOR FIVE-FACTOR SOLUTION

| Variable | Distance, Time | Personal Chars. | Primary Knowledge | Alternate Knowledge | Stress <br> Response |
| :---: | :---: | :---: | :---: | :---: | :---: |
| Mileage | 0.831 |  |  |  |  |
| Tasks, pre-depart. | 0.773 |  |  |  |  |
| Time. pre-depart | 0.662 |  |  |  |  |
| Penalties, late arrival | 0.543 |  |  |  |  |
| Age of youngest child |  | 0.909 |  |  |  |
| Age of commuter |  | 0.888 |  |  |  |
| Flexibility, arrival |  | 0.720 |  |  |  |
| Times seek info., pre-depart |  | 0.628 |  |  |  |
| Flexibility. departure |  | 0.590 |  |  |  |
| Gender of commuter |  | 0.498 |  |  |  |
| Street names in desc. |  |  | 0.863 |  |  |
| Landmarks in desc. |  |  | 0.817 |  |  |
| Actively seek info., pre-depart |  |  | 0.578 |  |  |
| Stress, pre-depart |  |  |  | 0.859 |  |
| Seeks info, pre-depart |  |  |  | 0.771 |  |
| Tasks, primary route |  |  |  | 0.647 |  |
| Change in stress, alt. routes |  |  |  | 0.625 |  |
| Number of alt. rtes. known |  |  |  | 0.617 |  |
| Stress, primary route |  |  |  |  | 0.822 |
| Modifications. primary route |  |  |  |  | 0.759 |

Thus, designers of effective information systems need to consider the distance traveled by and the time available to commuters, personal characteristics of the commuters, commuters' knowledge of their primary and alternative routes, and commuters' responses to stress. The distance-time factor indicates that, with increasing commuting distance, commuters have less time available before departure and they accomplish fewer significant tasks before departure. The personal characteristics factor indicates positive correlations among age, sex, age of the commuter's youngest child, flexibility of arrival and departure, and times the commuter seeks information before departure. The primary route knowledge factor shows the relationship between the detailed knowledge of street names and landmarks and the time at which commuters first seek out information regarding traffic conditions on their commute. The alternative-route knowledge factor demonstrates the intercorrelations of number of alternative routes known, stress when using an alternative route, stress experienced before departure, how actively commuters seek out their first traffic information, and the number of tasks performed on the commute. Finally, the stress response factor shows the relationship between the number of modifications made to the primary route and the amount of stress experienced on the average commute.

## Analysis of Responses to VMS Message Manipulations

As discussed earlier, accuracy of perception and probability of changing route in response to messages given by VMS were also studied. Results were analyzed using repeated-measures ANOVA. Values of the $F$-statistic and related probabilities are presented in Table 5.
Commuters were more likely to correctly interpret the message when presented with a specific task rather than a generic task. Further, they were more likely to correctly interpret the message when the reason was presented before the task. Interestingly, a pattern completely in opposition to the pattern just described was observed for the probability of commuters changing route in response to the message. Commuters indicated that they would be more likely to change their route when the message presented a generic (rather than specific) task and when the task (rather than the reason) was presented first. Finally, commuters indicated that they would be most likely to change their route in response to a message if the message presented a generic reason and did not present the task.
Task information appears of secondary importance to commuters. Further, commuters prefer generic reasons. This finding may indicate that commuters wish only to know that a

TABLE 5 RESULTS OF VMS REPEATED MEASURES ANALYSIS

| Variable | F | $\boldsymbol{p}$ |
| :--- | :--- | :--- |
| Correct response, gencric vs. specific task | $9.141(1,83)$ | 0.003 |
| Correct response, task first vs. reason first | $16.719(1,8.3)$ | 0.001 |
| Probability of route change, gencric vs. specific task | $141.142(1,89)$ | 0.001 |
| Probability of route change, task first vs. reason first | $66.726(1,89)$ | 0.001 |
| Probability of route change, gencric vs. specific reason, | $22.17 .7(1,94)$ | 0.001 |
| task present vs. task absent |  |  |

traffic problem exists and that they wish to tailor their response to their specific commuting goals (25). These findings also may be medium dependent in that the observed pattern of commuter responses would not be observed if, for example, the messages were delivered by radio. Further, they can only be generalized to information delivered en route, not to information delivered before departure.

## CONCLUSION

The survey and analyses described have produced a picture of an extremely complex commuting population, but one with definable needs that can be grouped parsimoniously, producing important implications for the design of motorist information systems. The method employed has produced information that would not have been available through use of a standard survey and has provided a set of baseline responses that will allow any changes or modifications to existing information systems to be examined for efficacy.

The analyses also raise a set of additional questions for researchers interested in motorist information systems. One question would involve applicability of these findings to other commuting corridors. Work is currently under way at the University of Washington to extend this method to studies of other corridors in the Seattle area. Studies of other major commuting groups using a similar method would allow for comparison of findings and a search for more general principles that could be followed in the design of information systems. A second question would involve the stability of the identified subgroups once they have received tailored motorist information. Thus, the results observed could well serve as a baseline for changes in the behavior of Seattle-area commuters that might be traced to delivery of motorist information.

From the discussion of the analyses, the central premisethat commuters cannot be considered as a homogeneous population - has been supported. The method employed focused on those differences and has identified aspects of commuters' daily tasks that help determine their use of and response to motorist information.

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[^0]:    Liberty Mutual Research Center, 71 Frankland Road, Hopkinton, Mass. 01748.

[^1]:    $1 \mathrm{p}>|\mathrm{t}|$, two-tailed significance probability
    ${ }^{2} p>F^{\prime}$, where $F^{\prime}$ is the ratio of the larger to the smaller
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[^5]:    - $=$ No significant difference in MOE.
    $\AA$ = Significant increase in MOE for second truck type (i.e., the second truck had more effect than the first truck on the oncoming vehicle).
    $=$ Significant decrease in MOE for second truck type (i.e., the second truck had less effect than the first truck on the oncoming vehicle).
    $\square=$ Insufficient sample size.

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[^7]:    *Light trucks defined as pickups, utility vehicles, vans, and other trucks not exceeding $10,000 \mathrm{lbs}$. gross vehicle weight.

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[^16]:    ${ }^{a}$ Significant treatment effect.

[^17]:    ${ }^{a}$ Significant treatment effect.

[^18]:    ${ }^{a}$ Significant treatment effect.

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[^22]:    * Estimate for the year 1984 from unpublished FHWA data and Bureau of Census

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