# 1213

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# *Human Performance and Highway Visibility*

## *Design, Safety, and Methods*

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# Transportation Research Record 1213

# Contents

# Foreword

Of the three basic components in the highway system the human has proven the most difficult to understand and model. This Record begins with papers that have been able to use human performance data and findings to create usable engineering models. Taoka found a statistical function that adequately models the probability density function of the surprise reaction times of drivers. The model output suggests revised reaction times for use by designers and traffic engineers. Michaels and Fazio developed a model of the freeway merge maneuver based on driver behavior. Using the model, engineers can estimate the speed change lane length necessary to ensure that the driver coming from a ramp would find a usable gap 85 percent of the time. Farber and Matle have revised the DETECT seeing-distance model to include additional driver characteristics. The model is now more accurate and is available in a personal computer version. Woltman and Szczech developed a methodology for comparing retroreflective signing materials in various placements, road geometrics, and distances.

Research on driver response to various elements of the highway system is important for better design and operation of the roadway system. The next four papers contribute to that knowledge base. Because color provides information to drivers, color rendition under artificial illumination is important. Hussain et al. tested various types of light sources and recommended those that result in acceptable color rendition. Zwahlen analyzed the detection distance of reflectorized targets at varying degrees of peripheral visual angle. The result is particularly important for sign design on vertical or horizontal curves. Comprehension of symbols used on signs is critical to their effectiveness. Paniati describes the evaluation, redesign, and retesting of 22 symbolic signs. Hummer also studied signing, using a simulator to evaluate supplemental signs about local attractions. Although the signs were generally detrimental to driver control of the vehicle, further study is needed. Driver speed control, particularly speed variability, was analyzed by Garber and Gadiraju in terms of highway geometrics and safety consequences.

Techniques to collect data on driver behavior and performance are important to furthering and improving research but are often useful for engineering studies of highway and traffic operations. The final two papers, which were presented at a mid-year symposium sponsored by Committees A3B02 and A3B06, describe advances in the state of the art in research techniques. Van der Horst and Godthelp discuss both in-vehicle and outside-the-vehicle techniques. Both types of instrumentation feature computer-based methods for quick, efficient, and flexible use. Allen et al. describe a system for using lap-top computers for in-car data acquisition and analysis. This system replaces bulky, conventional data collection equipment.

# An Analytical Model for Driver Response

GEORGE T. TAOKA

**An analytical model using the lognormal probability density function is applied to published driver response time measurements. Close agreement is obtained when this function is fitted to the measured responses of drivers to the onset of the amber signal as they approach signalized intersections. Statistical parameters associated with three sets of measured data are presented. Parametric estimates of 1.1 to 1.2 sec for the median, 1.3 to 1.4 sec for the mean, and 0.55 to 0.75 sec for the standard deviation of the distribution function appear to give the best fit to the experimental values.**

The statistical distribution of driver response time is an important consideration in traffic engineering and highway design. AASHTO uses a design reaction time value of 2.5 sec to determine stopping sight distances (*1*). The *Transportation and Traffic Engineering Handbook* (*2*) suggests a value of 1.0 sec for computing the yellow clearance interval at a signalized intersection. Parsonson, in a discussion to a paper by Wortman and Matthias (*3*), indicates that computation of the yellow clearance interval was a controversial issue during the preparation of the Handbook.

It is the purpose of this paper to present an analytical model that estimates the measured statistical distributions of three sets of published response time measurements. The need to estimate different percentile ranges of driver response times has been suggested recently by Shapiro et al. (*4*). They suggest that future editions of *The Manual on Uniform Traffic Control Devices* should include a section on design driver criteria. Design driver response times for braking should be broken down by the 50th, 85th, and 95th percentile categories of the driver population. They also suggest breaking down response times by key driver groups, such as by age or other significant driver characteristics.

Presented in this paper are key percentile estimates for response times associated with three sets of experimental data to which the lognormal probability density function was fitted.

## LOGNORMAL PROBABILITY DENSITY MODEL

The lognormal probability density model has been applied to represent driver reaction times measured under anticipatory conditions (*5*). The lognormal function was shown to give an acceptable fit, within the 5 percent level of significance, to reaction time measurements published by Moss and Allen in 1925; Johansson and Rumar in 1971; and by Gazis et al. in 1960 (*5*). In the first two investigations, drivers were forewarned that their responses were to be measured. In the third investigation, drivers were tested involuntarily.

University of Hawaii at Manoa, Honolulu, Hawaii 96822.

The lognormal function will be used to model the measured values of the surprise response times of drivers. The equation $f(t)$ of this probability density function is given by

$$f(t) = \frac{1}{\sqrt{2\pi}\,\xi t} \exp\left[-\tfrac{1}{2}\left(\frac{\ln t - \ln \lambda}{\xi}\right)^2\right] \qquad (1)$$

where $\lambda$ equals the median value of response time. If $\mu$ equals the mean value and $\sigma$ equals the standard deviation, then the measure of dispersion $\xi$ is given by the formula

$$\xi^2 = \ln\left(1 + \frac{\sigma^2}{\mu^2}\right) \qquad (2)$$

The cumulative probability distribution function $F(t)$ is given as

$$F(t) = \int_0^t f(\tau)\, d\tau \qquad (3)$$

The properties of this function are discussed by Ang and Tang (*6*).

## FITTING THE MODEL TO MEASURED DATA

In 1983, Wortman and Matthias published the results of measured response times of 839 drivers to the onset of the amber signal at signalized intersections (*3*). This is one of the largest samples of surprise response time measurements available in the literature.

The tests were conducted at six intersections in Arizona. Daytime values were obtained at all six intersections and nighttime response times were recorded at two of the six intersections. All drivers were tested involuntarily. Response time measurements appeared to be a function of the locations tested. For example, the mean values ranged from 1.09 to 1.55 sec and the 85th percentile estimates varied between 1.5 and 2.1 sec at the different locations. For the entire sample, the mean value was 1.3 sec and the standard deviation was 0.60 sec. The 85th percentile value was reported to be 1.8 sec. Using Equations 1 and 2, the model parametric values were determined to be $\lambda = 1.14$ sec and $\xi = 0.439$. The estimates of the statistical distribution from this model are shown in Figure 1 and summarized in Table 1.

The model was fitted also to data provided by Chang et al. (*7*). The 579 drivers in this study also were responding to the

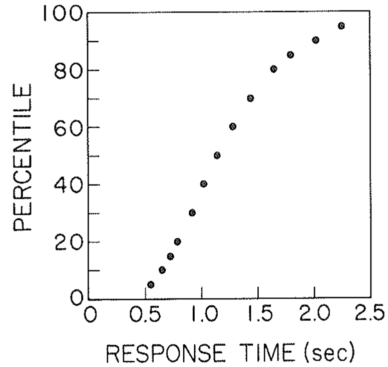


**FIGURE 1 Response times from Wortman and Matthias (3).**

onset of the amber traffic signal and were unaware that their response times were being measured. These measurements were recorded under diverse circumstances, including daylight and evening conditions, on both wet and dry roadways, and at peak and offpeak hours. The response time measurements therefore reflect the variability in lighting and weather conditions under which the investigation was conducted. The reported distribution times were

Median value $\lambda$ = 1.1 sec
Mean value $\mu$ = 1.3 sec
85th percentile = 1.9 sec
95th percentile = 2.5 sec

The computed parameter estimates for these data were

Standard deviation = 0.736 sec
Dispersion parameter = 0.527

The third set of data to which this model was fitted was published by Sivak et al. (8). The response values presented in this study were measured during car-following experiments in which the distance between a lead car and a follower car was varied from approximately one to five car lengths. The responses of the driver of the follower car to the appearance of the brake lights of the lead car were carefully recorded. These measurements were recorded under clear and sunny daytime conditions. The 87 drivers in this third sample therefore were tested under nearly ideal driving conditions. The reported values were 1.38 sec for the mean and 0.56 sec for the standard deviation. The computed parametric estimates were $\lambda$ = 1.19 sec for the median and $\xi$ = 0.390 for the dispersion parameter.

## COMPARISON OF THE PROBABILITY DISTRIBUTIONS

The median, mean, 85th, 90th, and 95th percentile estimates for the three sets of experimentally measured response times are given in Table 2. The median values fall between 1.1 and 1.2 sec, the mean values are between 1.3 and 1.4 sec, and the standard deviations are between 0.55 and 0.75 sec. There is

TABLE 1 DISTRIBUTION OF RESPONSE TIMES (3)

| Cumulative Distribution | Response Time (Sec) |
|---|---|
| 0.05 | 0.55 |
| 0.10 | 0.65 |
| 0.15 | 0.72 |
| 0.20 | 0.79 |
| 0.30 | 0.91 |
| 0.40 | 1.02 |
| 0.50 | 1.14 |
| 0.60 | 1.28 |
| 0.70 | 1.44 |
| 0.80 | 1.64 |
| 0.85 | 1.80 |
| 0.90 | 2.01 |
| 0.95 | 2.25 |

TABLE 2 PERCENTILE RESPONSE TIME DISTRIBUTION ESTIMATES FROM EXPERIMENTAL MEASUREMENTS IN SECONDS

| Source | Median | Mean | Standard Deviation | Percentile 85th | 90th | 95th |
|---|---|---|---|---|---|---|
| Wortman (*3*) | 1.14 | 1.30 | 0.60 | 1.80 | 2.01 | 2.35 |
| Chang et al. (*7*) | 1.10 | 1.30 | 0.74 | 1.90 | 2.16 | 2.50 |
| Sivak et al. (*8*) | 1.19 | 1.38 | 0.56 | 1.78 | 1.96 | 2.26 |

close agreement between the data of Wortman and Matthias (*3*) and the results of Sivak et al. (*8*) in the 85th to 95th percentile range.

The 85th to 95th percentile response times in Table 2 indicate that the shortest estimates are computed from Sivak's measurements, while the longest estimates are derived from Chang's data. These results can be explained by the fact that Sivak's testing was done under daylight conditions, while Chang's investigation was conducted in the evening and during the day and in both clear and inclement weather.

It should be mentioned that Chang et al. postulated the existence of a driver lag time interval between perception time and brake reaction time (*7*). The median value of driver lag time was estimated to be approximately 0.2 sec, with the 85th percentile estimate between 0.6 and 0.7 sec. The existence of driver lag time is not addressed in this paper. Further experimental study is required to clarify the issue. Sivak (*9*) has written an excellent survey article on driver reaction times in car-following situations.

## CONCLUSIONS

It has been shown that the lognormal function can be used to model the probability density function of the surprise response times of drivers. The median driver should respond within 1.2 sec, the mean driver within 1.4 sec, and the 85th percentile driver within 1.9 sec, under surprise braking conditions.

The reaction time of 1.0 sec suggested for use in the amber clearance interval formula (*2*) may be insufficient. A value between 1.2 to 1.8 sec, corresponding to the 85th percentile estimate, is suggested. The possibility of lengthening this design parameter should be investigated.

The AASHTO design value of 2.5 sec may correspond to the response time of the 95th percentile driver. The stopping sight distance design driver assumption is satisfactory at the present time.

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# Driver Behavior Model of Merging

RICHARD M. MICHAELS AND JOSEPH FAZIO

**A model of freeway merging is developed based on driver behavior. The model proposes that a ramp driver accepts a gap based on the first order motion vectors of an approaching vehicle, which perceptually is angular velocity. This function simultaneously accounts for relative velocity and distance separation. The total model includes driver behavior on the ramp, steering control onto and on the speed change lane, acceleration, gap search, and an abort zone. The model was tested on 102 merges on both a curved ramp and a tangent connector to the speed change lane. The results indicated that an angular velocity model did explain the merge decision point and that drivers used an angular velocity threshold criterion. Using the model, it was possible to estimate the speed change lane length necessary for the ramp driver to find an acceptable gap 85 percent of the time. This length increased with decreasing ramp design speed, but decreased with increasing volume. In general, a speed change lane length of 800 ft is sufficient to ensure an acceptable gap 85 percent of the time for all freeway volumes over 1,200 passenger cars/lane/hour and ramp design speeds over 30 mph, assuming an acceleration capability of the ramp vehicle of greater than 1.5 ft/sec$^2$.**

Historically, the design of acceleration lanes on freeways has been based on the acceleration characteristics of vehicles and empirical observations of merging behavior. Most of the evidence suggests that drivers begin to merge when the relative velocity difference between freeway gap vehicles is less than 5 mph. Current design practice is to define a speed change lane length (SCLL) for a given freeway design speed so that vehicles on the speed change lane (SCL) can achieve a speed difference on this order. The length depends on the ramp connector, its radius of curvature, and its grade. AASHTO policy defines SCLL with all these factors considered (*1*).

A more fundamental approach to defining SCLL is to start with the driving task. Michaels (*2*) suggested a simple model, which was extended by Drew (*3*). The question is: On what basis do drivers decide that a gap exists sufficient for them to steer into the freeway lane? If it were possible to define the analytic process used by drivers to reach this decision, it would be possible to define the SCLL. The purpose of this paper is to provide such a model and to test it empirically.

## MERGING TASK

The merging process requires the driver to perform a series of tasks. Although they are smoothly integrated by the driver, the tasks represent discrete steps that must be taken in appropriate order if the merge maneuver is to be successful. These tasks may be defined as follows:

1. Ramp curve tracking,
2. Steering transition from ramp to SCL,
3. Acceleration,
4. Gap search, and
5a. Steering transition from SCL to freeway or
5b. Abort.

These are shown in Figure 1.

The first two tasks involve a pursuit tracking process (*4*). It is hypothesized that most drivers will complete these tasks before a gap search phase begins. This is because pursuit tracking requires matching the angular velocity of the curve with the yaw velocity of the vehicle, which is a continuous process. This leaves little opportunity for timesharing with gap search. In addition, the geometrics of most curved ramp connectors give drivers insufficient sight distance to observe approaching vehicles on the adjacent freeway lane. Finally, with a normal downgrade on the ramp connector, the motion vectors of the adjacent roadway and vehicles on it involve second and third order motion. Most evidence in the psychological literature suggests that humans do not scale or use these higher order derivatives of motion (*5*). Thus, in this phase of the merging task, drivers do not have a stable basis for judging the speed or location of freeway vehicles, absolute or relative.

Once on the speed change lane, however, the driver is following a course parallel to the freeway. Steering control is now a compensatory task (*6*). Given the handling characteristics of most modern vehicles, the amount of time available for visual search is relatively large. Furthermore, assuming

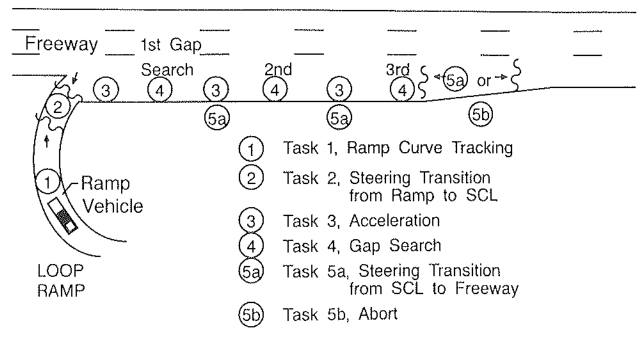


**FIGURE 1 Elements of the merge maneuver.**

Urban Transportation Center, University of Illinois at Chicago, Chicago, Ill. 60641.

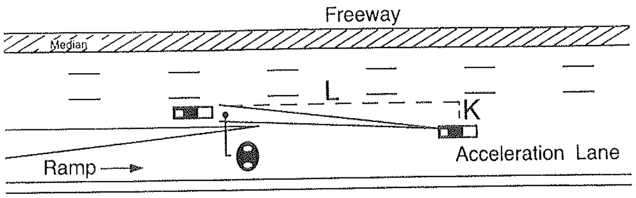


**FIGURE 2 Angular velocity.**

that the SCL is long relative to the time and space required for the merge, the driver can view the approaching traffic on the freeway lane. Thus, the driver is in the gap search phase.

In this phase, it is possible to define the velocity vectors of approaching vehicles relative to the driver on the SCL. This has been defined by Michaels (*2*) and Drew (*3*). The paradigm is shown in Figure 2. Essentially, the driver can evaluate the angular velocity, which is the first order motion vector relative to the ramp driver. This angular velocity is defined quantitatively by a simple first order differential equation:

$$w = k(V_f - V_r)/l^2 \qquad (1)$$

where

- $w$ = angular velocity (rad/sec),
- $V_f$ = freeway vehicle speed (ft/sec),
- $V_r$ = ramp vehicle speed (ft/sec),
- $l$ = distance separation (ft), and
- $k$ = lateral offset (ft).

This function is continuous, with $w$ depending jointly on the speed difference and separation. The function varies between plus and minus infinity.

It is hypothesized that drivers operate at some criterion level of $w$, that is, a threshold that will vary among drivers. If drivers adopt such a criterion as the basis for defining an acceptable gap, then there is a means of predicting the time or distance gap required for a merge maneuver. There is empirical evidence (*6,7*) that the angular velocity threshold is in the range of 0.01 to 0.001 rad/sec with a nominal value of 0.004 rad/sec.

As drivers observe an approaching vehicle on the freeway lane, the vehicle generates an angular velocity as a function of its speed relative to the ramp driver and its distance from that driver. Depending on the speed and distance relationship, the observed angular velocity will have one of three properties: (a) a positive value, (b) a negative value, or (c) a null value. The first case defines the situation in which the ramp vehicle is traveling at a speed greater than the freeway vehicle, that is, an opening condition. The second case defines the situation in which the ramp vehicle is traveling at a speed less than the freeway vehicle, that is, a closing condition. The third case defines the condition where the relative speed and distance generate an angular velocity below threshold. These conditions are shown in Figure 3. The defining function in Equation 1 is one with two variables determining angular velocity so that the curve shown is selected from the two-dimensional space.

In the first case, the driver is free to merge so long as there is sufficient lead headway. Thus, if the angular velocity of the *lead* vehicle is at or below threshold, the merging driver may steer onto the freeway lane. For design purposes, the second case is the critical one. If the speed of the ramp vehicle is less

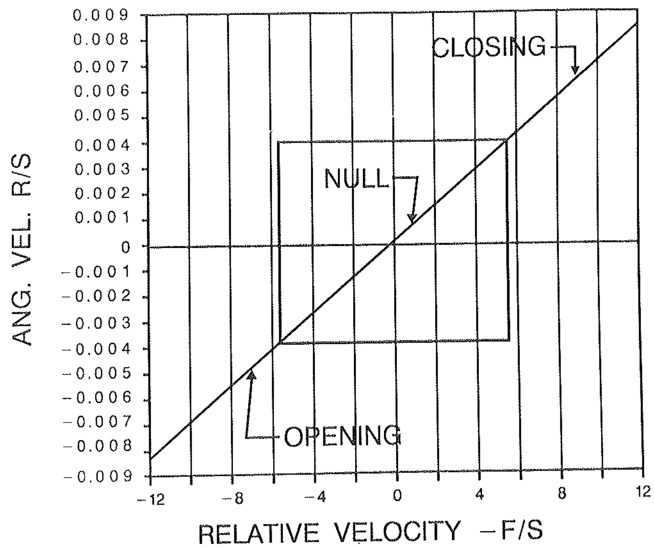


**FIGURE 3 The angular velocity function.**

than that of the approaching freeway vehicle and that of the lead freeway vehicle, then the decision to merge is determined wholly by the angular velocity of the approaching vehicle. It is hypothesized that in this case the merge will be determined solely by the angular velocity of the following vehicle with the limiting condition that over the length of the SCL the relative positions of the ramp and lead vehicles are such that the ramp vehicle is behind.

Given these conditions, it is possible to define the merge process and to predict where a merge will occur. When the ramp driver steers from the controlling curve and drives parallel to the freeway lane on the SCL, he/she looks down the freeway at approaching vehicles. If the observed gap, lead, and lag generate a less than threshold angular velocity, the driver initiates the steering maneuver required to merge. The sight distance must be sufficient at this initial gap search to estimate the angular velocity of an approaching vehicle. If the lag gap generates a supra-threshold angular velocity, the ramp driver will reject the gap. The control action available to the ramp driver is acceleration. Such acceleration leads to a smaller speed difference between the ramp and freeway vehicles. It is hypothesized that the ramp driver will not initiate a second gap search until the end of the acceleration phase. In other words, the hypothesis is that there is an iterative process: gap search, acceleration, gap search. In theory, this iteration continues until the ramp vehicle speed equals or exceeds freeway speed or the ramp vehicle runs out of SCL.

The best strategy for a ramp driver under congested conditions is to accelerate until the vehicle speed reaches that of the freeway. Unfortunately for ramp drivers, there is no stable criterion for the duration of acceleration required to reach a relative velocity difference of zero or greater. Nor is there a stable criterion for knowing whether the SCL is long enough to reach the relative velocity condition within the acceleration capabilities of a vehicle. A reasonably effective strategy is to use the iterative process described above. Thus, in congested freeway traffic, it is predicted that drivers will demonstrate a speed profile that is a step function as shown in Figure 4.

Merging behavior is constrained by two factors. One is the

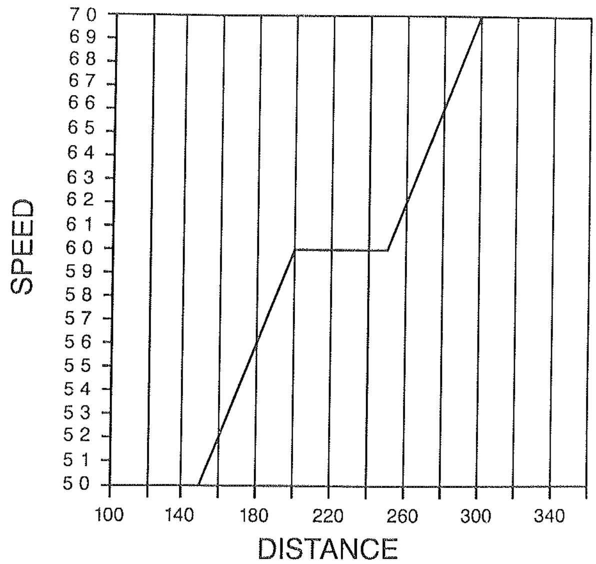


**FIGURE 4 Predicted ramp vehicle speed profiles.**

controlling ramp curvature, which determines the speed of the ramp vehicle when entering the speed change lane. In loop ramps this is determined by the design speed of the curve. In diamond ramps, this is determined by the grade and angle of convergence.

The second factor is the gap distribution function of the freeway traffic, which is volume dependent. The distribution of gaps can be defined by an Erlang distribution (*7*) and can be used to estimate the probability of occurrence of a gap meeting the angular velocity criterion for a ramp vehicle arriving at random at any point on the SCL.

Thus, a model of merging can be defined from driver behavior considerations. It is directly possible to validate the proposed model. This test is described in the next section.

## MODEL TEST

As described above, the angular velocity model defines the basic information required to test its validity. There are three fundamental questions that must be answered:

1. At the point where steering onto the freeway begins, is there a constant angular velocity between ramp and lag vehicles?
2. Is there an observable and consistent pattern of acceleration on the SCL, and is it related to the angular velocity at merge?
3. Is there an observable and consistent pattern of gap search iterations by ramp drivers?

To answer these questions, it is necessary to measure the behavior of ramp drivers and the properties of the lag gaps into which they enter. This was done by continuous video recording of both ramp and freeway vehicles. Since the primary interest was merging under heavy freeway volume conditions, data were collected during peak periods. Data were collected on both a loop ramp connector and a descending diamond ramp connector.

Measurement was made from a structure overlooking (and at some distance from) the subject SCL. The SCL was marked at 50-ft intervals for the first 500 ft and at 100-ft intervals thereafter. Both freeway and ramp vehicles were recorded for at least 1 hr during the peak period.

The videotapes were analyzed on a frame-by-frame basis. Therefore, it was possible to track ramp and freeway vehicles in 0.03-sec intervals. Because the distance along the speed change lane was known from the lane markers, it was possible to derive velocity and acceleration for ramp and freeway vehicles. The distance along the SCL where merge began was determined by the point where the left wheel of the ramp vehicle first crossed into the right-hand freeway lane.

To test the angular velocity model, it was essential to measure the location and speed of the freeway vehicle ahead of which the ramp vehicle merged. This was done by starting at the point of merge and determining the location of the lag vehicle on the freeway. It was then possible to rewind the tape and follow the lag vehicle to estimate its speed. It was also possible, using this procedure, to estimate the speed and position of the lead freeway vehicle relative to the ramp vehicle and the beginning of the merge maneuver. The analysis procedure allowed the tracing of the time-distance relations of the ramp vehicle along the SCL as well as these same relations for the freeway vehicles that bounded the accepted gap. Given these data, it was possible not only to obtain a velocity profile for the ramp vehicles but also estimate angular velocity at the beginning of the merge maneuver.

Because this was peak-hour traffic, vehicles entered the test ramp alone and in platoons of two or more vehicles. To account for any interactions, speed, position, and merge measurements were made for each vehicle in a group that was on the SCL simultaneously. Therefore, it was possible to examine the merge process for each vehicle in a queue.

There was a wide variation in the gap distribution on the right-hand freeway lane. Using a criterion of angular velocity threshold, the analysis was restricted to those merges where the lag vehicle was closer than three seconds, that is, to those cases where drivers had a significant merge choice.

## RESULTS

### Test of the Angular Velocity Model

The angular velocity at the point of merge beginning was computed given the speeds of vehicles on the ramp and the freeway and the separation between vehicles. The angular velocity was computed for both a loop ramp and a diamond ramp. In the case of the loop ramp, the distributions for one-vehicle, two-vehicle, and three-vehicle moving platoons were calculated. The median value was calculated as the best measure of central tendency. A nonparametric test was used to compare the different medians. No significant differences in angular velocity were found for vehicles due to platoon size. Consequently, all the data were pooled and the results are given in Table 1. The median angular velocity was found to be 0.0043 rad/sec. The variance of the distribution was found to be 0.0016 rad/sec. A similar analysis was carried out for a sample of vehicles entering on a diamond interchange. The results are given in Table 2. The median angular velocity at beginning of merge was 0.0034 rad/sec. Thus, the results are consistent and indicate that the decision to merge on the basis of the angular velocity criterion is reasonable.

TABLE 1 ANGULAR VELOCITY DISTRIBUTION FOR LOOP RAMPS

| ANGULAR VELOCITY (RADS/SEC) | FREQ. OF OCCURRENCE | CUMULATIVE % FREQ. |
|---|---|---|
| −.043 − −.032 | 1 | 1.2% |
| −.031 − −.020 | 2 | 3.5% |
| −.019 − −.008 | 10 | 15.1% |
| −.007 − +.004 | 28 | 47.7% |
| +.005 − +.016 | 21 | 72.1% |
| +.017 − +.028 | 8 | 81.4% |
| +.029 − +.040 | 4 | 86.0% |
| +.041 − +.052 | 1 | 87.2% |
| > +.053 | 11 | 100.0% |
| TOTAL | 86 | |
| MEDIAN = | 0.0043 | |

TABLE 2 ANGULAR VELOCITY DISTRIBUTION FOR A DIAMOND RAMP

| ANG. VEL. | FREQ. | CUM.% |
|---|---|---|
| −.008− −.004 | 1 | 0 |
| −.004 − .000 | 5 | 6.25% |
| .000 − +.004 | 6 | 37.50% |
| +.004 − +.008 | 1 | 75.00% |
| +.008 − +012 | 1 | 81.25% |
| +012 − +.016 | 2 | 81.25% |
| N = | 16 | 100.00% |
| | | MEDIAN = .0034 R/S |

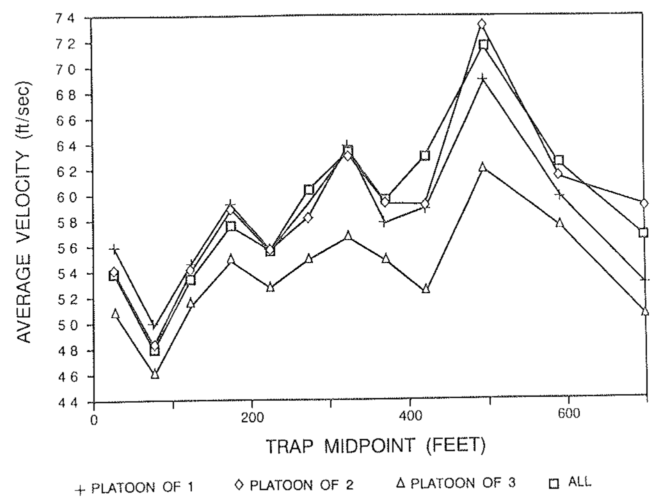


**FIGURE 5 Observed velocity profiles: Eastbound Lake Avenue to southbound I-94 (Edens).**

### Acceleration Patterns of Ramp Drivers

The velocity profiles of the merging vehicles from the loop ramp were plotted as a function of distance along the SCL. The averages are shown in Figure 5 for platoons of one, two, and three vehicles and for all combined. There does seem to be a series of steps. Unlike Figure 4, however, there is a decline in speed between successive accelerations rather than the hypothesized constant speed. This may be accounted for by the fact that drivers in this phase are attending to gap search, not speed control. A decline in speed may be expected for the duration of the gap search.

The number of these "steps" was counted for each ramp vehicle. It was possible to determine the percentage of gap search trials to merge. The data are presented below:

| *Trial* | *Distance (ft)* | *Proportion Merging* |
|---|---|---|
| 1 | 250 | 0.20 |
| 2 | 400 | 0.62 |
| 3 | 500 | 0.98 |

As indicated, 20 percent of the merges were made in one trial, 62 percent were made in two trials, and 98 percent were made in three trials. No more than three trials were observed in the data. If the speed of the ramp vehicle is known at the beginning of a gap search and the speed and volume on the freeway are known, using the Erlang distribution it is possible to calculate the probability that a gap will meet the angular velocity criterion during the first gap search. The probability of an acceptable gap for the subject SCL was 0.18, which is consistent with the observed proportion of 20 percent of first merges.

The data, although quite variable, are consistent with the model predictions. There are steps in acceleration followed by small decelerations that may be interpreted as incidental changes due to the driver attending to gap search. In addition, the probability of merging increases with successive trials. Almost all merges are made with three or fewer trials.

## DISCUSSION

The results of the empirical analysis confirm the proposed driver behavior model. The angular velocity of the approaching vehicle is the criterion that drivers use to make a merge decision. The value of 0.004 rad/sec is an acceptable threshold value. Drivers judge gaps in sequence, increasing the probability of finding one acceptable by accelerating between successive searches. The number of such searches depends on the speed-volume relations in the mainstream traffic, the acceleration capabilities of the ramp driver-vehicle combination, and the geometrics of the SCL.

In most geometric design situations, the perception of motion is based on relative or absolute angular velocity. Angular velocity determines the control response of the driver; it combines speed and position into a single metric. Furthermore, angular velocity explains traffic behavior under a variety of highway geometrics.

Although the data are consistent with the angular velocity model, the variability in the data is quite high. Measurement error and the enormous variability among drivers in the automobile-highway system account for the variability.

First, measurement error is significant in analyzing the video recordings. Each frame was projected on a video display terminal with a perspective grid overlay. A data analyst recorded when a subject vehicle crossed each of the distance markers, in sequence. Therefore, small time errors lead to large errors in calculating speed and acceleration. The farther down the ramp the vehicle proceeded, given perspective distortion, the greater the errors were likely to be. Not only were there individual errors in time-location, these errors propagated in estimating speed and acceleration. The longer a vehicle was on the SCL, the greater the errors were likely to be. These measurement errors add to the variability of calculated values such as angular velocity.

Second, the relative and absolute sources of speed and position information in the driving environment are very large. Although drivers use little second and third order information, these cues may reinforce expectations and responses. Moreover, angular velocity as the human measure of motion perception is on a continuum, that is, it is scaled according to the normal power law. Drivers need not operate on a conventional threshold criterion, but may scale the closure rate and select a gap acceptance criterion on an angular velocity well above the detection threshold. Furthermore, the system allows drivers to adopt other and mixed merge strategies. At one extreme, drivers learn that maximum acceleration on a SCL of reasonable length will always bring them to a relative velocity that will insure an acceptable gap. Drivers adopting such a strategy should have a speed profile that is continuous rather than discrete as the model suggests. Examination of individual speed traces shows such behavior. In the proposed model, it is hypothesized that the lag vehicle in a pair determines the merge decision. It is also possible to consider the lead vehicle. If a driver enters the SCL at a speed lower than the mainstream, the driver can track the angular velocity of the lead vehicle, accelerating to drive its angular velocity to threshold. At that point, regardless of separation, it would be safe to merge. Evaluating the lead vehicle angular velocities in the data suggests that some drivers operate in this fashion. None of the alternatives discussed is mutually exclusive and drivers are free to use any one or a combination.

Finally, the behavior of mainstream drivers is not passive. Drivers on the right lane of a freeway can detect and evaluate vehicles on the SCL. They can estimate their gap and determine whether it is sufficient for the merging driver to enter. They can then respond by slowing down to allow the merge or speeding up to prevent the merge. They can, given the uncertainty of ramp driver behavior, choose to change lanes to reduce the ambiguity. Such behaviors were observed. Although there is a tendency to treat traffic flow mechanistically, merging appears to be a highly dynamic human decision process and one that is often interactive.

All of these factors add to the variability in observed merge behavior. Although the variability is obvious, it is also clear that there is a consistent analytic process used by drivers in making a gap selection decision. This is the perceived angular velocity of the lag vehicle.

## SPEED CHANGE LANE LENGTH REQUIREMENTS

Given the behavioral model, it is possible to develop an SCLL estimating procedure. As a starting point, one can begin with the following assumptions:

1. A modern vehicle with acceleration capabilities in the range of 0.1 to 0.2 *g*,
2. No environmental restrictions on driver visibility,
3. Freeway volumes in the range of 1,200 to 2,000 passenger cars/lane/hour (pcplph),
4. Speed of entry onto the SCL determined by the ramp connector design speed, and
5. The SCL as a parallel lane tangent to the freeway without grade or curvature.

As previously discussed, the merge process is composed of four sequential decision components, to which a fifth component is added:

1. An initial steering control component,
2. An acceleration component,
3. A gap search component,
4. A merge steering component, and
5. An abort component.

Associated with each of these components is a length. The sum of these lengths is the total speed change lane length.

The initial steering control component may be derived from empirical work on steering (*4*). This is a constant time, which for normal drivers is 1 sec. Given the control ramp design speed, the required length of this segment can be calculated.

The initial acceleration component is defined as a time period of 2 sec in which the driver accelerates at a maximum rate of 0.15 *g*. This time period is derived simply by examining the sight distance available to the driver once on the SCL. Given a structure upstream from the SCL, a minimum distance is required before the ramp driver can see approaching vehicles in the freeway lane. Given a normal range of ramp design speeds, 2 sec of acceleration will provide the merging driver an unobstructed view of oncoming freeway traffic.

Gap search is the third component. Drivers need to view oncoming freeway traffic for 0.25–0.5 sec to detect angular velocity of a lag vehicle. Given the relative velocity at the end of the acceleration component, the probability that a gap will produce an angular velocity less than threshold, 0.004 rad/sec, may be estimated using the Erlang distribution. If the closing angular velocity of the lag vehicle is greater than .004 rad/sec, the ramp driver would reject the gap, accelerate, and then evaluate the new gap. It is suggested that the driver uses such an iterative process. However, there is no way to estimate the probabilities of each successive gap meeting the threshold criterion distribution using the Erlang.

Merge steering distance, the fourth component of the model, can also be computed. One way to approximate the total distance required for a ramp vehicle to accelerate to a speed such that the probability of finding an acceptable gap is 0.85 or greater is to rewrite Equation 1:

$$V_r = V_f \times (1 - [(w_t/k) \times V_f t_g^2]) \qquad (2)$$

The term $t_g$ is the 85th percentile gap length in time drawn from the Erlang for any given right-lane freeway volume, which, in turn, defines $V_f$ as the average speed of the freeway traffic. Given $V_r$ and knowing the ramp design speed, it is possible to calculate the distance required for the ramp vehicle to reach that speed assuming an acceleration of 0.15 *g* or 4.5 ft/sec². This length may be considered the gap search length.

Finally, the fifth length component is the abort distance. This is the length required for a driver who does not find a gap to decelerate and, if necessary, stop before running out of SCL. In essence, there is a length from the end of the SCL at which the terminus generates an angular velocity greater than 0.004 rad/sec. When a driver reaches this distance, he or she must either decelerate or make a forced merge. In any event, this length requires the driver to shift from gap search to an avoidance response. Equation 1 can also be used to estimate this length component. The ramp terminus is assumed to be a taper section. Hence the angular velocity of the taper diagonal is the cue for the driver. It is assumed in Equation 1 that the offset distance, $k$, is 4 ft. With the speeds derived from the above discussion and using $w_t$ = 0.004 rad/sec, Equation 1 can readily be solved for the length, $l$. In essence, a simple additive model has been defined to derive a SCLL that will allow drivers to find an acceptable gap 85 percent of the time for any freeway volume. The actual value is conditional on the gap distribution on the freeway, the acceleration acceptable to drivers of the ramp vehicles, and the angular velocity criterion used by drivers to define an acceptable gap.

Using the model, the SCLL was calculated for design speeds of controlling curves of 30 to 45 mph and freeway volumes ranging from 1200 to 2000 pcplph. The results are shown in Figure 6. These lengths are those such that the probability of an acceptable gap is greater than 0.85.

As may be seen, the SCLL decreases with increasing ramp design speed. What is most noteworthy is that SCLL decreases with freeway volume. The reason for this is simply that the freeway speed declines with volume. The mean speeds used in this analysis were taken from the *Highway Capacity Manual* (*8*). This explains the very short SCLL needed as freeway volume approaches practical capacity. The total SCLL derived from the model varied little over the range of freeway volumes from 1200 to 1800 pcplph and ramp design speeds from 30 to 45 mph. At a freeway volume of 1200, a 50 percent increase in ramp design speed leads to only a 20 percent reduction in required SCLL. For design purposes, if one used the length defined by the 1200 pcplph curve, the SCLL needed for most currently used ramp design speeds is between 650 and 800 ft. It is interesting to note that accident rates on acceleration lanes tend to reach a minimum at a length of 700 ft (*9*).

These lengths are conservative. They assume far more mechanistic behavior on the part of drivers than is actually observed. If a driver carries out the gap search process on

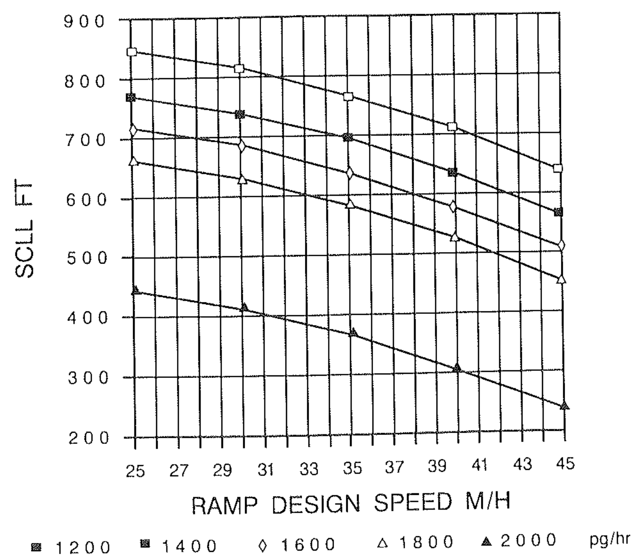


**FIGURE 6 Speed change lane lengths by ramp design speed for 85th percentile gap acceptance.**

each succeeding iteration, the relation between ramp driver and freeway gap is changed. More importantly, drivers can adjust speed and location relative to any gap and position themselves dynamically.

If the ramp connector is a tangent rather than a curve, the computation of the SCLL is simplified. If the angle of convergence between the ramp and SCL is small, that is, less than 3°, the steering control response required for transition is small and largely compensatory. It may be neglected in the model. In this case, the ramp becomes an acceleration component of the SCL. If drivers accelerate on the ramp at 0.1 to 0.2 *g*, their speed will be significantly higher at the point where they have a clear view of the freeway traffic than if on the loop connector, which precludes acceleration until physically steering onto the SCL. Therefore, the effective speed of the ramp vehicle at the time of first gap search is significantly higher in the diamond interchange case. Knowing the length of the connector and the normal acceleration used by drivers, it is possible to estimate the speed at the first gap search. The probability of finding an acceptable gap can then be calculated using the procedure discussed above.

The assumptions made in the above analysis are based on a descending ramp connecting with a flat tangent SCL. The analysis does not reflect the range of possible design situations, for example, an ascending ramp connector or an SCL with vertical curvature. What these design variations mainly influence is sight distance and vehicle acceleration. In the case of diamond type connectors, for example, descending ramps lead to higher speeds on entry to the SCL while ascending ramps lead to lower speeds. Adjusting the SCLL is straightforward if the grade and acceleration capabilities of the vehicles are known or are able to be estimated.

## SUMMARY AND CONCLUSIONS

The present study attempted to place the freeway merging process in a driver behavior framework. It used an angular velocity criterion for a driver's decision to merge. By defining the complete merge process as a five-step sequential task, the length of a parallel acceleration lane required for merging can be predicted.

The model was tested using two different types of ramp connectors, a curve and a tangent. Individual vehicles were tracked in time and distance from the end of the controlling curve to merge using high resolution video recording. This allowed the estimation of distance, speed, and acceleration of the ramp vehicle. It was possible to determine the angular velocity at the beginning of the merge maneuver by calculating the speed of the ramp vehicle at merge and that of a lagging freeway vehicle. Analysis of 102 vehicles in the two different design cases demonstrated a constant angular velocity at merge. This threshold value was consistent with previous research. Hence, the results of the research confirm that drivers do use angular velocity as a perceptual basis for the merge decision.

A procedure for estimating speed change lane length to insure acceptable gaps was developed using the model derived in this research. The procedure was applied under ideal design conditions using the worst case of a curved ramp connector to the SCL. The required SCLL was found to be independent of freeway volume over a range of 1200 to 2000 pcplph. The nominal SCLL to insure 85 percent or more merge opportunities for ramp drivers was no more than 800 ft.

The model also indicates clearly that a tangent ramp connector with a small angle of convergence leads to a more effective merge process. This is because the ramp becomes a part of the SCL on which drivers may accelerate prior to the gap search phase. The probability of an acceptable gap will be significantly higher than if the driver must steer from the ramp onto the SCL prior to acceleration, as is the case with a curved ramp connector.

## ACKNOWLEDGMENT

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# PCDETECT: A Revised Version of the DETECT Seeing Distance Model

EUGENE FARBER AND CALVIN MATLE

**Described in this paper is a revised version of the Ford Motor Company DETECT seeing distance model. The revised model, known as PCDETECT, is written in QuickBASIC for IBM-compatible personal computers. PCDETECT calculates the distances at which a driver can see various objects on the road at night as illuminated by the headlamp system specified by the user. The revised algorithms are based on Blackwell's recent contrast sensitivity research. They include new formulations for calculating contrast thresholds and take into account driver age, target size, background luminance, and individual differences. The revised model also incorporates a driver age factor for calculating veiling glare. The seeing distances calculated using the old and revised versions are generally in close correspondence. However, at low illumination levels, the new algorithm predicts seeing distances that are as much as 12 percent greater than the original version. This can be traced to differences between the old and revised contrast threshold functions. The age and variability factors in the new algorithm have a substantial impact on seeing distances. Against a low beam glare source at 300 ft, the seeing distance to a pavement edgeline was 413 ft for an average 20 yr old and 130 ft for a 15th percentile 70 yr old.**

In 1976 the Ford Motor Company published the results of a nighttime field study to validate the use of human contrast sensitivity functions to predict seeing distance to objects illuminated by motor vehicle headlights (*1*). We found that the seeing distance predictions based on these functions were generally in accord with the actual seeing distances measured in our field studies. As a result of these tests, we proceeded with the development of a computer seeing distance model, written in FORTRAN for mainframe application, which has become known as DETECT.

Described in this paper is a revised version of DETECT, written in Microsoft QuickBASIC for IBM-compatible personal computers. The revised version is known as PCDETECT. The development of PCDETECT was undertaken for several reasons. The primary reason was to incorporate the newer and more comprehensive vision algorithms that have become available in the literature since the original version of DETECT was developed. A second reason was to produce a more comprehensible and better documented program. The original FORTRAN version of DETECT is largely undocumented and is very difficult to maintain and modify. The new version has revised and thoroughly self-documented codes. Also, we felt that a program written for a personal computer would be accessible to more users than a mainframe version. Finally, we wanted to develop a more interactive and user-friendly interface to specify and vary systematically the input conditions.

Ford Motor Company, Dearborn, Mich. 48121.

Microsoft QuickBASIC was chosen as the development language because it is powerful and easy to use. QuickBASIC is a fast, compiled version of BASIC for IBM-compatible personal computers that incorporates advanced modular programming features and produces stand-alone machine language files.

In the rest of the paper, DETECT refers to the original mainframe FORTRAN version of the model and PCDETECT refers to the revised version written in QuickBASIC.

## HOW PCDETECT WORKS

DETECT and PCDETECT are headlamp seeing distance models. They use human contrast sensitivity formulations to calculate the distance at which various types of objects (referred to as "targets"), illuminated by headlamps, first become visible to approaching drivers. In DETECT, the only targets are pedestrians and longitudinal lane lines. PCDETECT deals with eight types of targets, as follows:

1. Longitudinal lane lines,
2. Transverse lane lines (e.g., pedestrian walkway markings),
3. Other pavement markings (e.g., words, symbols on surface),
4. Reflective pavement markers,
5. Traffic signs,
6. Post-mounted delineators,
7. Freestanding markers (e.g., traffic cones), and
8. Pedestrians.

The definition of "visibility" depends on the target. For all target types except 3 and 5, the visibility distance is the distance at which the driver is first able to *see* the target as a separate object. No assumptions are made regarding the relationship between seeing and recognition. The algorithms assume an attentive driver. For traffic sign targets, PCDETECT calculates both the seeing distance to the sign panel itself and the legibility distance of the sign elements, that is, letters or symbols. For type 3 targets, the visibility distance criterion is also legibility.

As background for a discussion of the revisions, it is useful to review the workings of the model in its present version. The core of PCDETECT is an algorithm for determining the threshold luminance contrast between an object and its background. The threshold contrast is the contrast at which the object is just discernible to an attentive observer.

Both DETECT and PCDETECT use an iterative procedure to increase and decrease the distance between the observer's vehicle and the target until it finds the distance at which the target is at the threshold, that is, is just visible to the observer-driver. However, in DETECT the observer's location is fixed and the target is moved; in PCDETECT, the target is fixed and the observer's vehicle is moved. In both versions of the model, the distance between the observer's vehicle and the glare source is held constant throughout the iteration process. This means that in PCDETECT, the glare car moves back and forth with the observer's car during the distance iteration process. PCDETECT also differs from the older version in that PCDETECT provides the option of multiple glare vehicles whose distances from the observer's are based on traffic volume.

The procedure for determining whether or not the target is visible at each "trial" distance is outlined below.

First, the model calculates the total candlepower from all of the vehicle headlamps falling on the target. The luminance of the target is then given by

$$L_t = R \sum_{i=1}^{n} (CP_i/D_i^2) \quad (1)$$

where

$R$ = reflectance of the target,
$CP_i$ = incident candlepower from the $i$th headlamp, and
$D_i$ = distance between the $i$th headlamp and the target.

Background luminance (assuming that the effective background is the road surface or a sign panel) is similarly calculated. For more distant backgrounds, the luminance is assumed to be an ambient value unaffected by the headlamps.

It is also necessary to determine the size of the target. Size is defined as the angle subtended at the observer's eye by the diameter of a visually equivalent disk. Blackwell defines the equivalent disk as a disk having the same area as the projected area of the target.

To make these calculations it is necessary to have the candlepower tables for the particular set of headlamps to be represented in the model, the dimensions of the target, the locations of the vehicle and the target on the road with respect to some common coordinate system, and the geometry of the road itself, that is, the horizontal and vertical curvature.

Once target and background brightness have been determined, the target-background contrast is calculated. Contrast is defined as

$$C = \frac{|L_t - L_b|}{L_b} \quad (2)$$

where $L_t$ and $L_b$ are target and background luminance values, respectively.

The effect of glare from opposing vehicles is represented by adding the veiling luminance ($B_v$) to the denominator of Equation 2:

$$C = \frac{|L_t - L_b|}{L_b + B_v} \quad (2a)$$

This contrast is then compared with the threshold contrast, that is, the contrast required for detection, calculated by PCDETECT's visibility algorithm. The distance is increased or decreased according to whether the actual contrast is greater or less than the threshold detection contrast. The actual and required contrast values are computed at the new distance and the process continues until the ratio of the actual and threshold contrast values reaches some criterion level. The distance at which this occurs is the "seeing distance."

These visibility calculations are described in the next section.

## DETECT VISIBILITY ALGORITHM

The visibility algorithm in the original version of DETECT is based on threshold contrast curves published by Blackwell in 1952 (*2*) and shown here in Figure 1. Threshold contrast is plotted as a function of background luminance for disk targets of various diameters. The threshold contrast increases with decreasing luminance and decreasing target size, that is, less contrast is required to see large objects against a bright background than small objects against a dim background. The curves in Figure 1 are for targets exposed to the observer for 1/30 sec. This particular set of curves is incorporated into DETECT in the form of empirically fitted equations.

The curves in Figure 1 are based on data from only two observers. However, DETECT incorporates adjustments to these curves for age and individual differences, which are based on data from a large sample of observers (*3*). The data on individual differences are used to estimate seeing distance percentiles, i.e., the distance at which some percentage of drivers will be able to see the specified target.

The calculation of disability glare in DETECT is based on Fry's veiling glare equation (*4*). This equation incorporates no adjustment for driver age.

### PCDETECT and the "19/2" Model

Blackwell's visibility research results, as they apply to highway illumination, are summarized in Publication 19/2.1 of the Commission Internationale de l'Eclairage (CIE) (*5*). The CIE

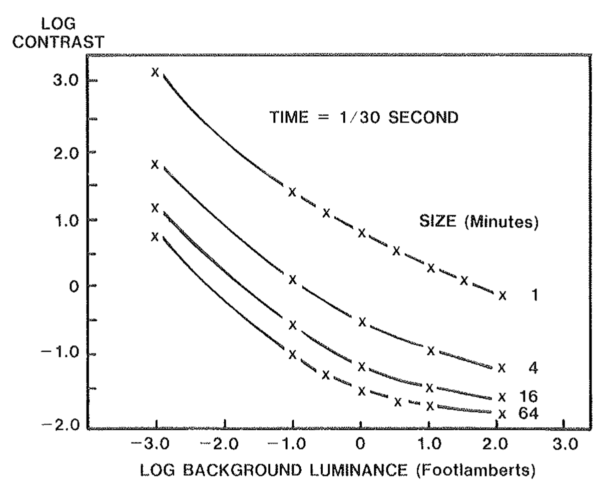


**FIGURE 1 Log threshold contrast as a function of background luminance for different size targets.**

report tries to organize his findings to evaluate the adequacy of roadway illumination. The whole system as it is presented in CIE 19/2.1 is not suitable for inclusion in a headlamp seeing distance model. However, many of the underlying formulations are applicable and it is these with which we are concerned. These newer formulations are based on laboratory research conducted by Blackwell over a 10-yr period using several hundred observers of varying age and visual capability.

As noted above, the original version of DETECT uses an empirical fit to the 1952 contrast threshold data, which has no theoretical basis. By contrast, CIE 19/2.1 presents an analytical expression for contrast sensitivity that has its roots in vision theory. The form of the equation used to calculate threshold contrast in PCDETECT is as follows:

$$C_{\text{th}} = cx \frac{0.0923}{n} \left[ \left( \frac{S}{tL_e} \right)^{0.4} + 1 \right]^{2.5} \qquad (3)$$

where

$C_{\text{th}}$ = threshold contrast
$cx$ = target size factor
$S$ and $t$ = parameters that depend on observer age and target size (discussed below),

$L_e = L_b + B_v$, the effective background luminance, and

$$n = \left[ \left( \frac{S}{100t} \right)^{0.4} + 1 \right]^{2.5} \qquad (4)$$

Figure 2, taken from the CIE report, is a plot of Equation 3 for a 4-min disk target and a 22-yr-old observer.

The $S$ parameter in Equation 3 is a function of both age and target size:

$$\log S = 0.5900 - 0.6235 \log d - s \qquad (5)$$

where $d$ is target size (diameter) in minutes of arc and $s$ is an age-related factor (see below).

The effect of the $S$ parameter is shown in Figure 3, which is a plot of the relative contrast sensitivity (RCS) function. The RCS function is the inverse of the threshold contrast function shown in Figure 2, normalized to have a "reference" value of 1.0 and 100 cd/m². The $S$ parameter changes the *shape* of the curve, as shown in Figure 3, but not the reference value.

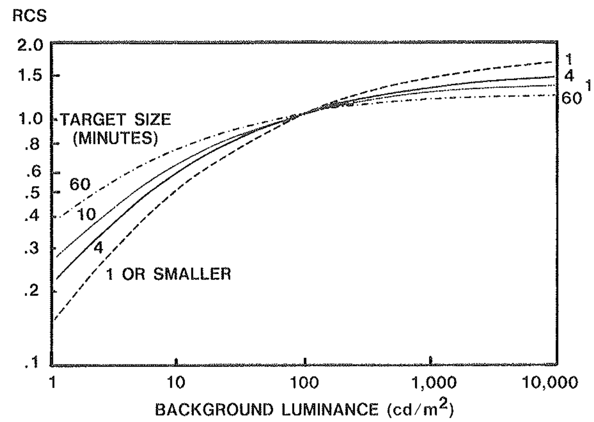


**FIGURE 3 The effect of target size on relative contrast sensitivity.**

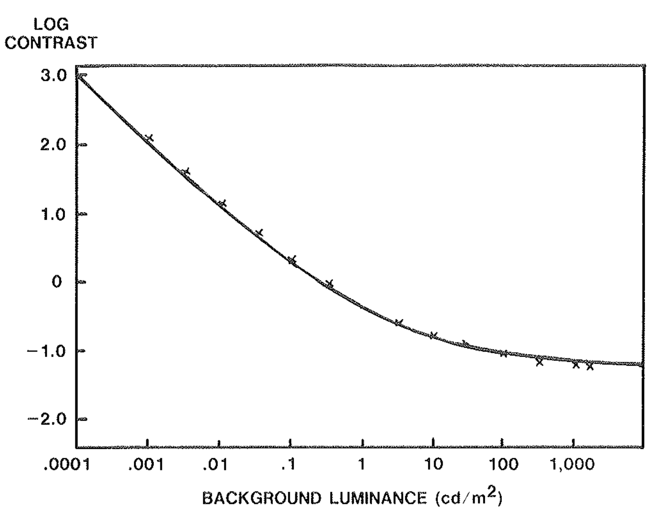


**FIGURE 2 Log threshold contrast as a function of background luminance.**

### Target Size in PCDETECT

In Blackwell's new system, target size is regarded as a visual task difficulty factor for which an empirically determined contrast multiplier must be supplied. In the original version of DETECT, the effect of target size was built into the equations to produce a near exact fit to the family of curves shown in Figure 1. In PCDETECT, a separate algorithm was developed to generate a size factor, $cx$, as follows:

$$d \leq 10: \quad cx = 3(0.37)^{\log_2 d}$$

$$d > 10: \quad cx = 0.106 - 0.0006d \qquad (6)$$

where $d$ is target size in minutes.

The algorithm expressed in Equation 6 was developed to produce approximately the same effect of size on the magnitude of the threshold contrast as the algorithm in the original version of DETECT. Figure 4 shows the contrast threshold function used in the DETECT model (shown in Figure 1) compared with the size-adjusted thresholds produced by

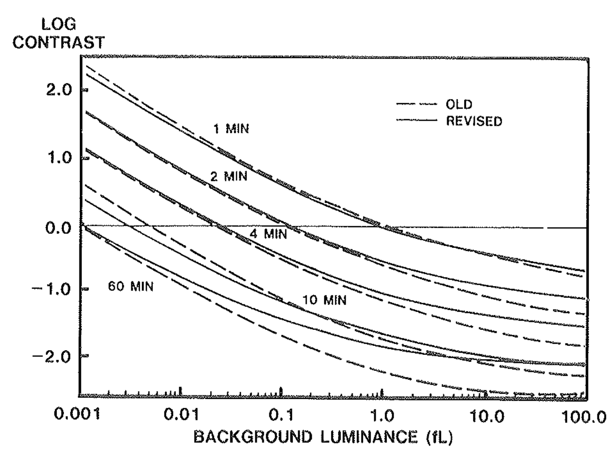


**FIGURE 4 Log threshold contrast function for different target sizes using old and revised algorithms.**

Equation 3. Note that while the two sets generally conform below 0.1 fL, they tend to separate at higher values because the newer curves generated by Equation 3 are somewhat flatter than the old curves, especially for larger target sizes. At low levels of background brightness, and for targets between 10 and 60 min, the new algorithm gives lower thresholds. This leads to differences in seeing distance predictions between the old and the new algorithms, as we shall see below.

In Blackwell's paradigm, the size of noncircular objects is defined as the diameter of a circle having the same projected area as the object, expressed in minutes of arc. In DETECT, size is defined as the diameter of a circle with an equal perimeter. This definition was used because it gave a better fit to the field data collected in the Ford validation studies (*1*) than the equal-area definition. Both algorithms are incorporated into PCDETECT; however, at present, only the perimeter algorithm is carried out. This algorithm is used to calculate size for all targets except letters and symbols. For letters, symbols, and other graphic elements, both on vertical surfaces and on the road surface, the target size is the stroke width of the element in minutes.

### Age and Individual Differences

In PCDETECT, the age of the observer affects both the shape of the threshold contrast curve and the magnitude of the threshold. Earlier it was mentioned that the $s$ and $t$ parameters, which determine the shape of the threshold function, are age-related. The functions for $s$ and $t$ are given in CIE 19/2.1, as follows:

Age 20–44: $s = 0$

44–64: $s = 0.00406\,(A - 44)$

64–80: $s = 0.0812 + 0.00667\,(A - 64)$ (7)

Age 20–30: $\log t = 0$

30–44: $\log t = -0.01053\,(A - 30)$

44–64: $\log t = -0.1474 - 0.0134\,(A - 44)$

64–80: $\log t = -0.4154 - 0.0175\,(A - 64)$ (8)

where $A$ is observer age in years.

Contrast threshold values increase considerably with age. Figure 5, taken from Blackwell (*3*), is a scatter diagram of the log threshold contrast values obtained from 156 observers at a background luminance of 0.01 fL, which is a typical pavement luminance several hundred feet from the vehicle. Note that the effects of age and individual differences are roughly comparable. Contrast sensitivity decreases by a factor of about five between the ages of 20 and 80. The age effect is represented by a contrast multiplier, which Blackwell refers to as $m_1$. The function, as given in CIE 19/2.1, is as follows:

Age 20–42: $m_1 = 1.000 + 0.00795\,(A - 20)$

42–64: $m_1 = 1.175 + 0.0289\,(A - 42)$

64–80: $m_1 = 1.811 + 0.1873\,(A - 64)$ (9)

The variability of thresholds also increases with age. The

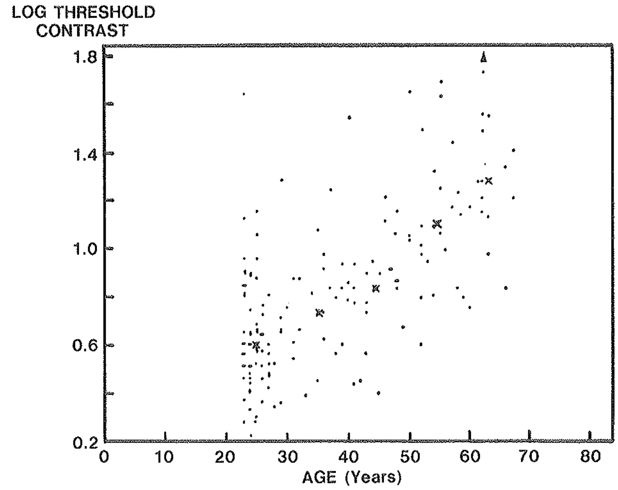


**FIGURE 5 Log threshold contrast as a function of age at 1 fL background luminance.**

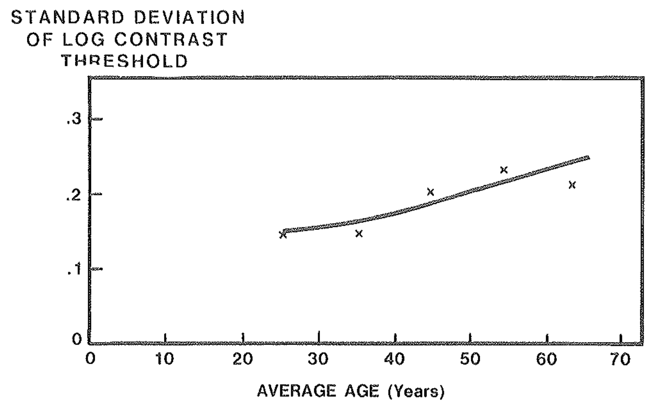


**FIGURE 6 Observer variability as a function of age.**

expression for the relationship between age and variability of the threshold was derived from data summarized in Figure 6, taken from Blackwell (*3*), and is as follows:

Age ≤ 35: $\sigma_{\log} = 0.124 + 0.001133\,A$

> 35: $\sigma_{\log} = 0.064 + 0.002850\,A$ (10)

where $A$ equals driver age in years and $\sigma_{\log}$ is the standard deviation of the log contrast threshold values.

Variability also increases with decreasing background luminance. Accordingly, a correction factor is applied to the log standard deviation. The correction factor was developed from the relationship shown in Figure 7, also taken from Blackwell (*3*). The correction factor is as follows:

$\log L_b \leq -0.5$: $cf = 1.0875 - 0.065 \log L_b$

$\log L_b > -0.5$: $cf = 1.012 - 0.216 \log L_b$ (11)

where $L_b$ is the background luminance and $cf$ is the correction factor. The correction factor is multiplied by the standard deviation of the log threshold contrast ($\log C_{th}$) to obtain the corrected value for the standard deviation.

The combined result of the age and luminance effects on variability is substantial. For example, the standard deviation

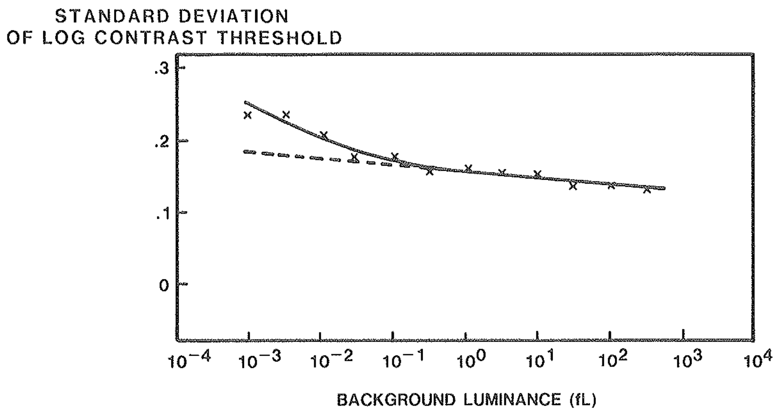


**FIGURE 7 Variability among 20- to 30-yr-old observers as a function of background luminance.**

of the log threshold contrast for a 60-yr-old observer at 0.01 fL is more than twice that for a 20-yr-old observer at 10 fL.

The probability distribution of contrast thresholds is log-normal (*3*), which permits the use of normal curve analytical tools for dealing with variation. A contrast multiplier is used to adjust the threshold to represent an observer at a given percentile level of performance. The adjustment factor is given by

$$cmp = 10 z_p \, \sigma_{\log C_{th}} \tag{12}$$

where

$cmp$ = contrast multiplier,
$\sigma_{\log C_{th}}$ = corrected standard deviation of the log-normal threshold distribution, and
$z_p$ = $z$-score (standard normal variable) associated with the $p$th percentile.

For example, the $z$-score associated with the 95th percentile is about 1.65. If the observer age is 35 and the background luminance is 0.3 cd/m², then from Equations 11 and 12, the corrected standard deviation is 0.184 and log $cmp$ is 0.303. The antilog of this value, 2.01, is the contrast multiplier associated with the 95th percentile level. From Equation 8, the value of $m_1$ for a 35-yr-old observer is 1.12. To accommodate 95 percent of 35 yr olds, the threshold computed from Equation 3 must be increased by a factor of $2.01 \times 1.12 = 2.25$.

## Disability Glare

The formula for calculating disability glare ($B_v$) used in the original version of DETECT is the Fry formula (*4*):

$$B_v = k \pi \sum_{i=1}^{n} \frac{E_v \operatorname{Cos}(\theta)}{\theta(\theta + 1.5)} \tag{13a}$$

where

$k$ = 10,
$n$ = number of glare sources (e.g., lamps),
$E_v$ = illumination from the glare source at the observer's eye, and
$\theta$ = observer's line of sight angle between the glare source and the target, measured in degrees.

The Fry formula includes no adjustment for driver age (*6*). Blackwell recommends an algorithm based on more recent and comprehensive data that does include a correction factor to reflect increasing sensitivity to disability glare with age. Veiling luminance ($L_v$ in Blackwell's notation) is given as follows:

$$L_v = k \sum_{i=1}^{n} \frac{E_v \operatorname{Cos}(\theta)}{\theta^2} \tag{13b}$$

where $k$ depends on driver age and the other parameters are as described for Equation 13a.

Both versions are programmed into PCDETECT. At present, Equation 13a is carried out to maintain consistency with the original version of DETECT. However, whereas the value of $k$ is fixed at 10 in the original version, in PCDETECT $k$ takes on different values depending on driver age, as described below.

Glare sensitivity, as expressed by the magnitude of the $k$ factor, increases with age. Blackwell's results on the effect of age on glare sensitivity are reported in a 1980 paper (*6*). Figures 8 and 9 show the factors, $m_3$ and $m_4$, by which $k$ increases with age for background luminance of 100 and 1.7 cd/m², respectively. Note that age increases the glare sensitivity by a factor of about 2.5. The value of $k$ in Equation 12b is 10 $m_3$ or 10 $m_4$ depending on the value of $L_b$. In

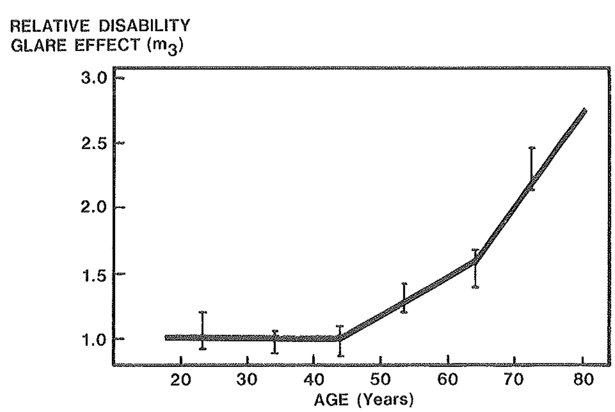


**FIGURE 8 Disability glare contrast multiplier as a function of age for background luminance values near 100 cd/m².**

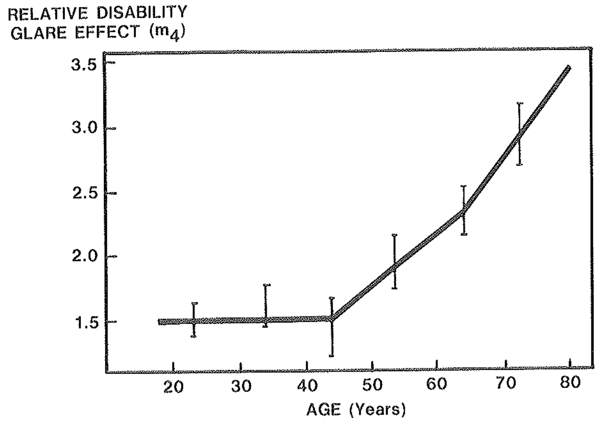


**FIGURE 9 Disability glare contrast multiplier as a function of age for background luminance values near 1.7 cd/m².**

PCDETECT, $m_3$ is used for values of $L_b$ equal to or greater than 3.8 fL and $m_4$ is used for smaller values.

The analytical expressions for $m_3$ and $m_4$ provided by Blackwell in CIE 19/2.1 are as follows:

$$m_3 \begin{cases} \text{Age} < 44\text{:} & m_3 = 1.000 \\ \text{Age 44–64:} & m_3 = 1.000 + 0.0301\,(\text{age} - 44) \\ \text{Age} \geq 64\text{:} & m_3 = 1.620 + 0.0725\,(\text{age} - 64) \end{cases}$$

$$m_4 \begin{cases} \text{Age} < 44\text{:} & m_4 = 1.5 \\ \text{Age 44–64:} & m_4 = 1.500 + 0.0419\,(\text{age} - 44) \\ \text{Age} \geq 64\text{:} & m_4 = 2.338 + 0.0668\,(\text{age} - 64) \end{cases} \tag{14}$$

In the older procedure, the veiling glare ($B_v$, as calculated by Equation 13a) is added to the denominator of the contrast expression (Equation 2a). PCDETECT follows the newer procedure of calculating a disability glare factor, which is subsequently used as a contrast multiplier. This gives the same results mathematically as the older procedure. The disability glare factor (DGF) is then given by

$$DGF = \frac{L_b}{L_b + vbf \times B_v} \tag{15}$$

where $vbf$ is the veiling brightness factor, given by

$$\theta < {}^1/_2 \quad vbf = 30.0$$

$${}^1/_2 < \theta < 1 \quad vbf = 59.0 - (58.0)\theta$$

$$\theta > 1 \quad vbf = 1.0$$

and $\theta$ is the glare angle.

The basis for the veiling brightness factor is discussed below.

### Glare at Small Angles

In the original version of DETECT, the threshold contrast for delineation targets is increased by a factor of 30 whenever glare from an opposing vehicle is present. This field factor is applied in addition to the veiling glare effect. This adjustment was necessary to reconcile predicted seeing distances with the actual seeing distances obtained in the Ford research (*1*). In the field research, center lines were the only delineation targets studies with glare, and the glare source used with this target was a pair of high beams. The combination of a small glare angle and intense glare source makes this a very severe glare condition. Nevertheless, in DETECT the 30-fold adjustment factor is used for all delineation targets when glare is present, no matter how distant or dim the glare source and no matter where the target is located laterally. No such field factor was needed in DETECT for pedestrian targets seen against glare; the veiling glare algorithm was sufficient.

The need for the adjustment factor for delineation probably is because of the small glare angle. Unfortunately, the available glare algorithms are not valid for glare angles less than about 1. It is difficult to study glare effects at small angles and the available research does not address glare angles below 0.75. Evidently, the effect can be considerable at small angles, as the Ford field research indicates.

Based on these considerations, PCDETECT uses a somewhat different procedure. The assumptions are made that (a) the additional visual task difficulty represented by the field factor is due to small glare angles and (b) the effect should be expressed as an increase in $B_v$ rather than as a contrast multiplier. Accordingly, a veiling brightness factor is used that has the value of 1.0 for glare angles larger than 1 and 30.0 for angles less than 0.5 with a linear ramp between 0.5 and 1. This is the algorithm shown in Equation 15. At low glare angles and high levels of $B_v$, this algorithm produces about the same effect as the 30-fold contrast multiplier used in DETECT.

### Calculating the Visibility Level

Visibility level ($VL$) is defined as

$$VL = C/C_{th} \tag{16}$$

where $C$ is the actual contrast between the target and its background and $C_{th}$ is the threshold contrast as predicted, for example, by Equation 3.

This form is correct only for a 20 yr old, 50th percentile observer with no glare. To take age, percentile, and glare into account we need to apply the contrast multipliers described above, as follows:

$$VL = 1/m_1 \times 1/cmp \times DGF \times C/C_{th} \tag{16a}$$

This is the expression used in PCDETECT. When $VL = 1$ the object is by definition at the borderline of detectability or legibility. At higher values of $VL$ the object becomes increasingly visible. Under a given set of conditions, an observer is increasingly likely to detect it or, in the case of sign elements, to find it legible.

The threshold contrast values predicted by Equation 3 represent laboratory conditions and may under- or overpredict the contrast required for reliable detection under field conditions. Some field adjustment factor will probably be necessary to apply the prediction to conditions involving more complex tasks and distractions of a real-world environment. The PCDETECT input menus allow the user to supply such a field factor in the form of a criterion visibility level ($CVL$), which differs from the nominal visibility level of 1.0 by some multiple. In order for the target to be regarded as detectable, the value of $VL$ would have to be equal to or greater than $CVL$, i.e., detection takes place when $VL = CVL$. A $CVL$ of 2.0, for example, would mean that the target is regarded as being visible when the contrast is twice the threshold contrast.

DETECT builds in field factors to reconcile calculated seeing distances with the actual seeing distances measured in Ford's field research (*1*). A factor of 0.2 is used for pedestrians. This means that the threshold contrast for pedestrian targets is ⅕ as great as that predicted by the 1952 1/30-sec Blackwell curves (*2*). In PCDETECT, this field factor is built into the target size scaling algorithm given in Equation 6. An additional field factor of 0.2 is applied for delineation targets in both versions of the model, i.e., delineation targets require 1/25 the contrast predicted by the 1/30-sec curves for threshold visibility.

## ROAD GEOMETRY

PCDETECT has a revised set of road geometry routines for dealing with horizontal and vertical curvature. These routines determine the locations and orientation of the target, the observer, and the observer and glare vehicle lamps with respect to a common coordinate system to calculate headlamp illumination levels and glare. Horizontal and vertical curvature are specified off-line by the user in a file that is referenced as part of the input process. The information can be taken directly from highway engineering blueprints.

PCDETECT also includes a routine for checking lines of sight over hill crests to determine when a crest vertical curve obscures an observer's view of the target or cuts off the line of sight from headlamps to the target or from a glare source to the observer.

## INPUT CONDITIONS AND THE PCDETECT USER INTERFACE

PCDETECT allows a very complete definition of the roadway visibility environment. This is necessary because there are many factors that influence seeing distance and thus need to be taken into account. The user must specify the headlamp beam patterns and configuration of the observer and glare vehicles; the characteristics of the driver; the geometry of the roadway, including lane geometry and horizontal and vertical curvature; the nature, characteristics, and location of the target on the roadway; and ambient luminance and reflectance values. Altogether there are more than 40 parameters that must be specified. One of the major objectives in developing PCDETECT was to make it easy to create, save, retrieve, and modify the input conditions for a given model run or series of runs. PCDETECT accomplishes this via a menu-driven user interface. This interface allows the user to create a new data set or retrieve a previously created set from disk storage. When creating an input data set, the user is lead through a series of input menus that provide reasonable default values for most of the parameters. Figure 10 is an example of such a menu. The user has the option of accepting the default values or entering new values. A new or old input data set can be run, modified, run again, and/or saved at the option of the user. One option is to develop a disk file with multiple input data sets. This allows the user to create and save a series of systematically modified input data sets in the same disk file. When the user elects to run this file, all of the input data sets will be run, one after the other.

## OUTPUT

At present, all output from PCDETECT is directed to the screen. Figure 11 is an example of the output. This screen is displayed with each distance iteration as the observer car is moved back and forth in search of the threshold. The screen displays the current estimate of the seeing distance, values for target and glare candlepower, the visibility level, veiling glare, and other parameters. The user is prompted after each iteration for a keypress to continue. Distance iterations continue until the visibility level differs from the criterion visibility level by less than 2 percent *or* the current seeing distance estimate differs from the previous estimate by less than 2 ft.

## SEEING DISTANCE PREDICTIONS WITH THE ORIGINAL AND REVISED ALGORITHMS

Model runs were made using the old and revised algorithms to calculate seeing distances to a 100-ft long, 4-in wide pavement line having a contrast of 1.0 with the pavement. The runs were made at different illumination intensity levels. A 20-yr-old observer was assumed. Both left-lane edge and right-lane edge lines were modeled. At the same distance, the incident candlepower is about four times higher on the right-lane edge than the left. The results are shown in Figure 12. In general, the correspondence between the two sets of predictions is very close. At high relative intensities, the older algorithm predicts somewhat longer seeing distances than the revised version, but the reverse holds at lower intensities. This is consistent with the difference in the contrast sensitivity curves as shown in Figure 4. The newer algorithm gives lower thresholds at low luminance levels for targets approximately 10 min or larger. The line target studied has an equivalent size of 10 min at 210 ft.

The model was run with the new algorithm to show the sensitivity of seeing distances to glare, age, and percentile level. The results are shown in Figure 13, which shows seeing distances, with and without glare, as a function of age for drivers with 15th and 50th percentile contrast sensitivities. The target was an 8 percent reflectance pedestrian, located 1 ft to the left of the right-edge line, and the glare vehicle was at 300 ft. Seeing distances declined with age, with the rate of the decline increasing with age. The decrease in seeing distances from age 20 to age 70 ranged from 45 percent to 59 percent under the four conditions. The aggregate effect of age, glare, and percentile level is considerable. Under the conditions simulated here, seeing distance ranges from 413 ft for a 50th percentile, 20-yr-old driver without glare to 130 ft for a 15th percentile, 70 yr old with glare. The rate at which seeing distance decreases with age is greater without glare, despite the fact that glare sensitivity increases with age. The reason is that the effect of glare is less at greater background

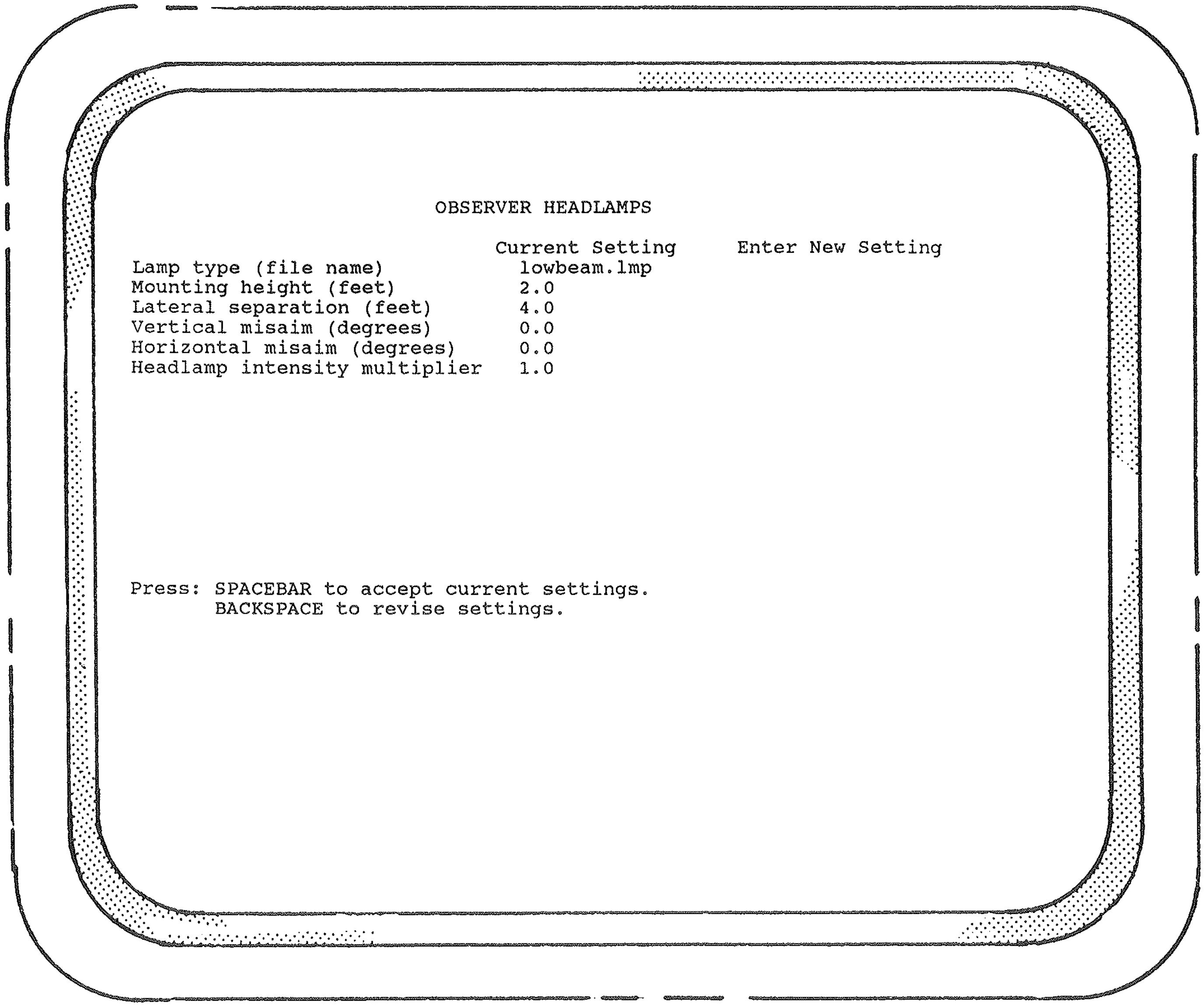


**FIGURE 10 PCDETECT input menu for headlamp characteristics.**

brightness levels and glare angles than at shorter seeing distances.

## VALIDATION

Blackwell's contrast sensitivity paradigm was applied to night vision with headlamps and validated in a general way in the Ford research mentioned earlier (*1*). In that study, seeing distances to delineation and pedestrian-shaped targets were determined in nighttime field research and compared with predicted seeing distances based on the Blackwell formulations. The various field factors and adjustments discussed above were used to tune the DETECT model. Further fine tuning of DETECT was done by selecting target definitions that gave the best fit to the data. Once these adjustments were made, the correspondence between field data and predicted seeing distance values was good, with the predicted values generally falling within one standard deviation.

PCDETECT has not been validated in a separate field study. However, PCDETECT was "tuned" to give results close to those of DETECT in situations common to both models.

We believe that the DETECT and PCDETECT algorithms based on this research are accurate enough to be useful. However, it has been approximately 14 yr since the Ford validation study was conducted and that research was not extensive. For example, only 12 subjects of varying ages were used in the study; target and background luminance values were estimated rather than measured for some targets; and the range of test conditions studied was very limited. Seeing distance to pavement lines against glare was measured only for the center line and against high beams as the glare source. In analyzing the results, the average performance of the 12 subjects was used and no attempt was made to account for age effects. Subjects were not "calibrated" to determine their contrast sensitivity in a laboratory setting. As Figure 5 shows, age effects and individual differences in contrast sensitivity are substantial. For all of these reasons, we believe that addi-

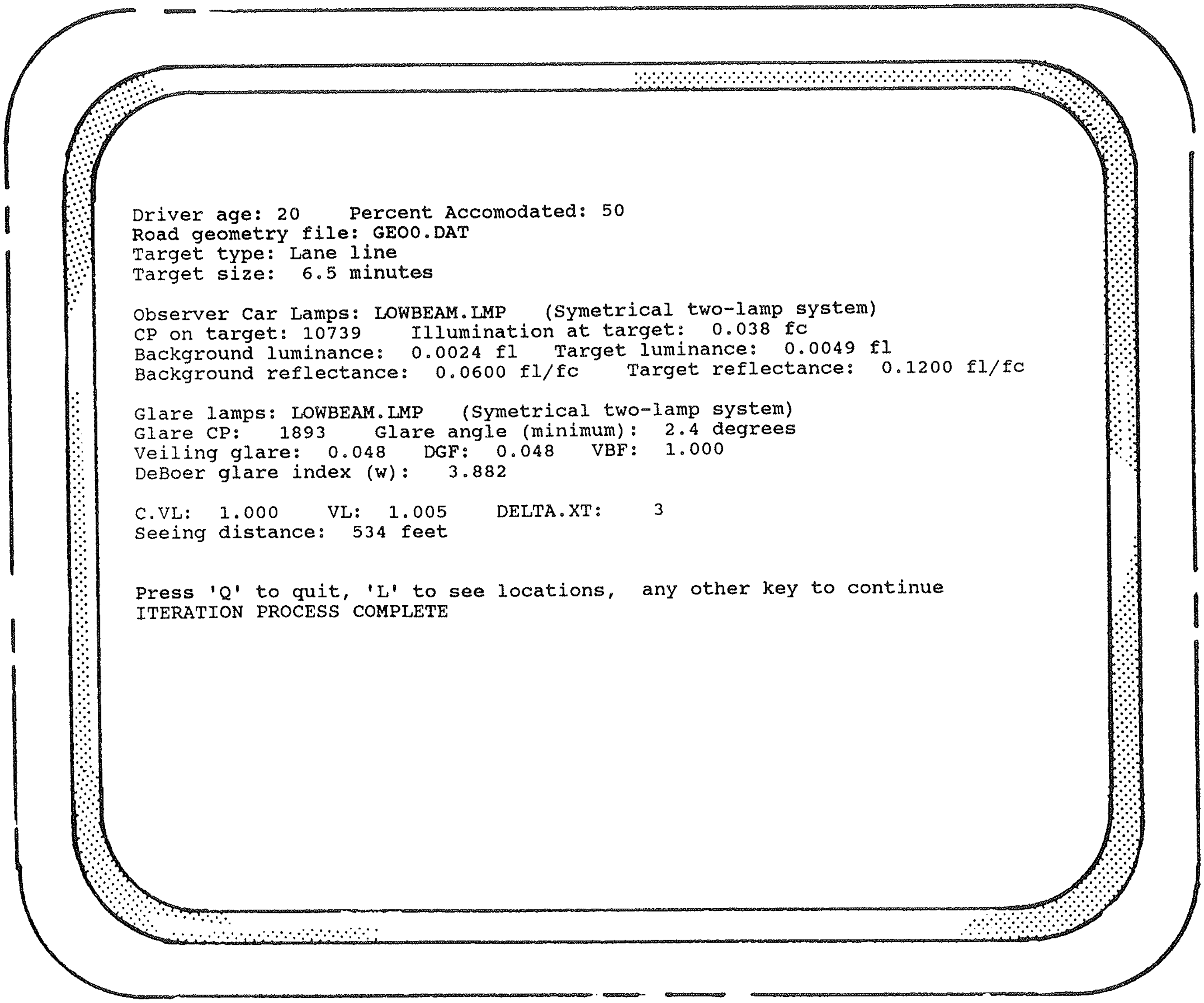


**FIGURE 11** Example of PCDETECT output screen.

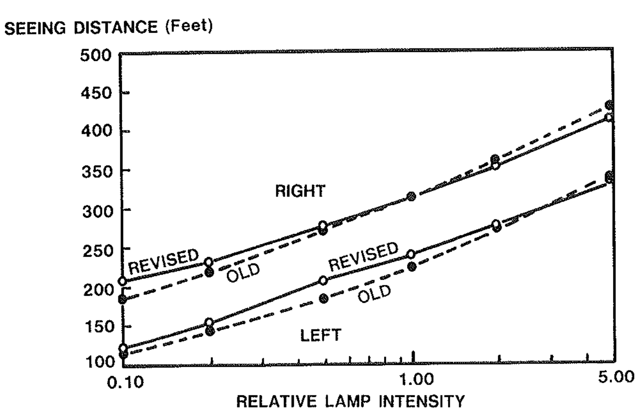


**FIGURE 12** Seeing distance to left- and right-side lane lines as a function of lamp intensity using old and revised algorithms.

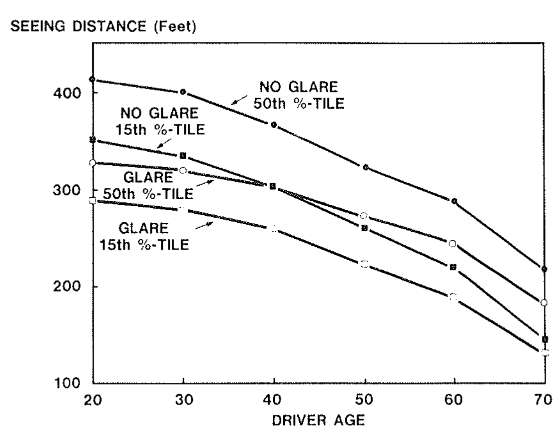


**FIGURE 13** Seeing distance as a function of age with and without glare for 50th and 15th percentile drivers.

tional work is needed to substantiate more thoroughly the field factors and assumptions used in both DETECT and PCDETECT.

We would very much like to see other oganizations with an interest in headlamp seeing distance models address some of these issues. It is probably not necessary to use large subject sample sizes or to engage in extensive nighttime field work. A more efficient alternative is to use a smaller number of carefully "calibrated" subjects, that is, observers whose contrast sensitivity has been established in the laboratory. Some of the issues can be addressed in detail in the laboratory and the results validated subsequently in the field. An example of such an issue is defining the equivalent Blackwell target for pavement lines. Validation would then consist of demonstrating that there is a general transfer function for predicting field performance from laboratory data.

We will be happy to consult with researchers who have interests along these lines. We invite comments from all interested parties regarding our implementation of the Blackwell formulations and the algorithms based on them that are described here.

## APPLICATIONS AND LIMITATIONS OF PCDETECT

PCDETECT (and the original version of DETECT) can determine the relative performance of different headlamp systems, expressed in terms of seeing distance, under some conditions. It can also determine the relative effect on seeing distance of varying such input parameters as misaim, glare intensity, driver age, and target reflectance. For such applications, it is sufficient to use representative or typical values of the fixed input parameters. However, PCDETECT cannot be used to calculate an accurate estimate of the seeing distance to a given object unless correct values for all of the parameters that define the target and viewing conditions are known, including the contrast sensitivity and glare susceptibility of the observer. Even with all important physical parameters held constant, the seeing distance for a given driver will vary from instance to instance because of underlying variability in sensory performance. Also, PCDETECT assumes a highly vigilant driver-observer, i.e., a driver who is looking for the visual target. Such high levels of vigilance are not sustained in actual driving. On the average, the seeing distance of a driver who has been alerted to look for a specific object is about twice that of a driver who is not expecting the object (*7*).

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# Sign Luminance as a Methodology for Matching Driver Needs, Roadway Variables, and Signing Materials

H. L. WOLTMAN AND T. J. SZCZECH

**The widespread use of retroreflective materials for information, regulation, and warning signs and the inclusion of retroreflective materials in many official standards suggest that a framework of luminance standards for minimum visual performance be adopted. Such a construct assumes that a variety of signing materials are available from which predictions of performance may be made. A methodology is provided to compare signing materials in a variety of placements, road geometries, and distances for existing types of retroreflective materials. The study compares the performance of retroreflective materials for existing headlamps and under many circumstances of use. Information is also provided on allowances for such factors as complex nighttime surroundings, the unalerted driver, and the relative importance of sign priority.**

Retroreflective materials enhance the nighttime visibility of traffic control signs and other devices. Various official standards require that signs which must be seen by a motorist at night be either retroreflecting or illuminated. Retroreflectorization alone is sufficient for sign visibility under reasonable conditions. These conditions include satisfactory alignment of the vehicle with the sign, an uncluttered surround to permit timely discovery, headlights in satisfactory alignment, and the use of specified retroreflective materials (*1–5*). Sign perception also depends on an adequate level of luminance. There are numerous factors that alter both the luminance capability of the retroreflective material and the adequate level of luminance required by the driver.

The factors that determine the luminance capability of retroreflective materials are highly mechanical. They have to do with headlamp output, choice of retroreflective material, and alignment of the sign with respect to the road. The factors that influence the level of luminance deemed adequate for the driver deal exclusively with the driver's perceptual process and state of mind. Therefore, a research methodology that explores a variety of scenarios representative of actual use may be a more satisfactory descriptor of the retroreflector than retroreflectance, its traditional descriptor.

The use of luminance as a criterion for evaluating performance of signs instead of retroreflectance provides a means to directly match driver needs. Estimates of luminance to satisfy driver needs can be obtained from a number of investigations. Driver needs from a review of nighttime sign legibility studies by Sivak (*6*) are presented below:

| *Level* | *Sign Legend Luminance ($cd/m^2$)* |
|---|---|
| Optimal | 75 |
| Replacement | |
| 85th percentile | 16.8 |
| 75th percentile | 7.2 |
| 50th percentile | 2.4 |

Studies dealing with Stop sign conspicuity by Morales (*7*), sign conspicuity for various background complexities and driver expectancy by Olson (*8*), and estimates for sign priority by Perchonok (*9*) show that satisfactory performance depends on sign luminance.

The level of luminance depends on sign position, roadway approach, and headlamp quality. It is correlated with such factors as retroreflectance, the weathered state of the sign, and cleanliness. Direct inspection of signs or reference to luminance tables that predict performance assures this necessary quality.

To determine the effects of vehicle and roadway variables on sign luminance, we have employed our previous findings of sign luminance for United States guide sign legends and backgrounds and the luminance enhancement from stream traffic and rainfall. These assessments used cars operating on both low and high beams and measured the luminance intensity of a variety of retroreflective materials from the position of the driver's eye in standard size passenger vehicles. Samples were taken in typical sign positions, from distances corresponding to the longest decision sight distance models to relatively short sign-reading distances. Headlamps used were either typical of new vehicle equipment or were supplied by equipment manufacturers following photometric testing. Aim was adjusted to correspond to SAE recommendations, usually employing the aiming screen method (SAE J 599). Level tangent sections of roadway were used. A full description of the method is contained in three separate papers by Youngblood and Woltman (*10–12*).

The findings are well-suited for adaptation to the problem at hand: Retroreflective materials will provide the same response curve given similar vehicle dimensions. Luminance values are proportional to illuminance, so that an accurate

3M Company, St. Paul, Minn. 55148.

comparison may be made between headlamps. Beyond this relationship, careful characterization of the retroreflective materials is required, as is the headlamp/driver-eye relationship. Angularity of signs with respect to the approach should be considered. Allowance for dirt on signs is the same as for dirt on headlamps; both are treated as a diminution of illuminance.

The procedure used in this study to model sign luminance was first detailed by Elstad et al. (*13*), with further refinement by Szczech (*14*). The model uses a detailed headlamp output (of the Westinghouse 6014) in a matrix encompassing all directions of interest for sign positions at any distance. The values derived for sign luminance involve complex geometric and retroreflective response relationships. Nevertheless, they correspond with the previously cited field studies and permit comparisons of sign luminances for three types of retroreflective materials, over a selection of limited alignment conditions.

## SIGNING MATERIALS

The signing materials studied were representative of new white retroreflective materials used for traffic control signs. Luminances for other colors and their ratios to white may be expected to fall within the following limits:

| *Color* | *Luminance Ratio to White (%)* |
|---|---|
| Yellow | 61 to 76 |
| Orange | 33 to 42 |
| Red | 17 to 30 |
| Green | 13 to 19 |
| Blue | 7 to 10 |

The materials studied are described below.

| *Reflective Sheeting* | | | *Coefficient of Retroreflection (cd/lux/m²) at 0.2° Observation −4° Entrance* |
|---|---|---|---|
| *Material* | *Type* | *Color* | |
| A | Enclosed lens | White | 120 |
| B | Encapsulated lens | White | 310 |
| C | Microprism | White | 1100 |

The coefficients of retroreflection, which are essential for sign luminance computations, are determined according to ASTM E 810-81.

## SIGN POSITIONS

Sign positions are shown in Figure 1. The positions are typical of regulatory and warning signs commonly displayed on the right shoulder; overhead signs over the driver's lane of travel; and signs on the left side, such as No Passing Zone pennants and bridge end barricades. The offsets and elevations used are specified in the *Manual on Uniform Traffic Control Devices* (*15*).

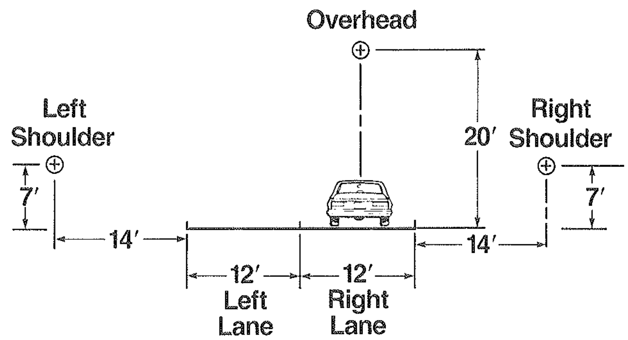


**FIGURE 1 Sign positions.**

## ROADWAY

The approach to the sign is not always a straight, level tangent section. It is frequently a horizontal or vertical curve. Five cases were chosen that, although they are an incomplete scenario, are representative and illustrative of a variety of approach conditions. These roadway geometries are diagrammed in Figure 2. They include a straight tangent approach, right and left curves with 2,000-ft radii, a sag, and a hill with a 6,000-ft radius. Curves of significantly smaller radii could not be considered because of the limitations of the headlamp matrix used for the computations.

## SIGN LUMINANCE ESTIMATES

Estimates of sign luminance for the three retroreflective materials at six approach distances and three sign positions are given in Tables 1 through 5. The values presented are for ideal conditions and do not have allowances or reductions for atmospheric transmissivity or windshield losses. These may be from 2 or 3 percent to 30 percent, respectively, for clear atmospheres and normal windshields. Allowance for weathering of signing materials is not included. The tables are arranged for separate roadway geometrics. The data presented in Tables 1 through 5 are further illustrated in Figures 3 through 6 to show the effect of road curvature, sign placement, and material type.

Two estimates are provided for signs seen at angles. Approximately 6 percent of the entrance angles of over 1,300 signs (*16*) are seen by the motorist at an angle of 30° or greater. Estimates are provided for shoulder signs angled 30° away from and 30° toward the road in Tables 6 and 7, respectively. It must be noted that the photometrics of the retroreflective materials are reasonably accurate for the entrance and observation angles actually encountered on all of the above approaches.

## DISCUSSION OF RESULTS

Tables 1 through 7 present estimates of sign luminance for distances from 366 m (1200 ft) to 61 m (200 ft) in increments of 61 m (200 ft). The luminance estimates are for a motorist

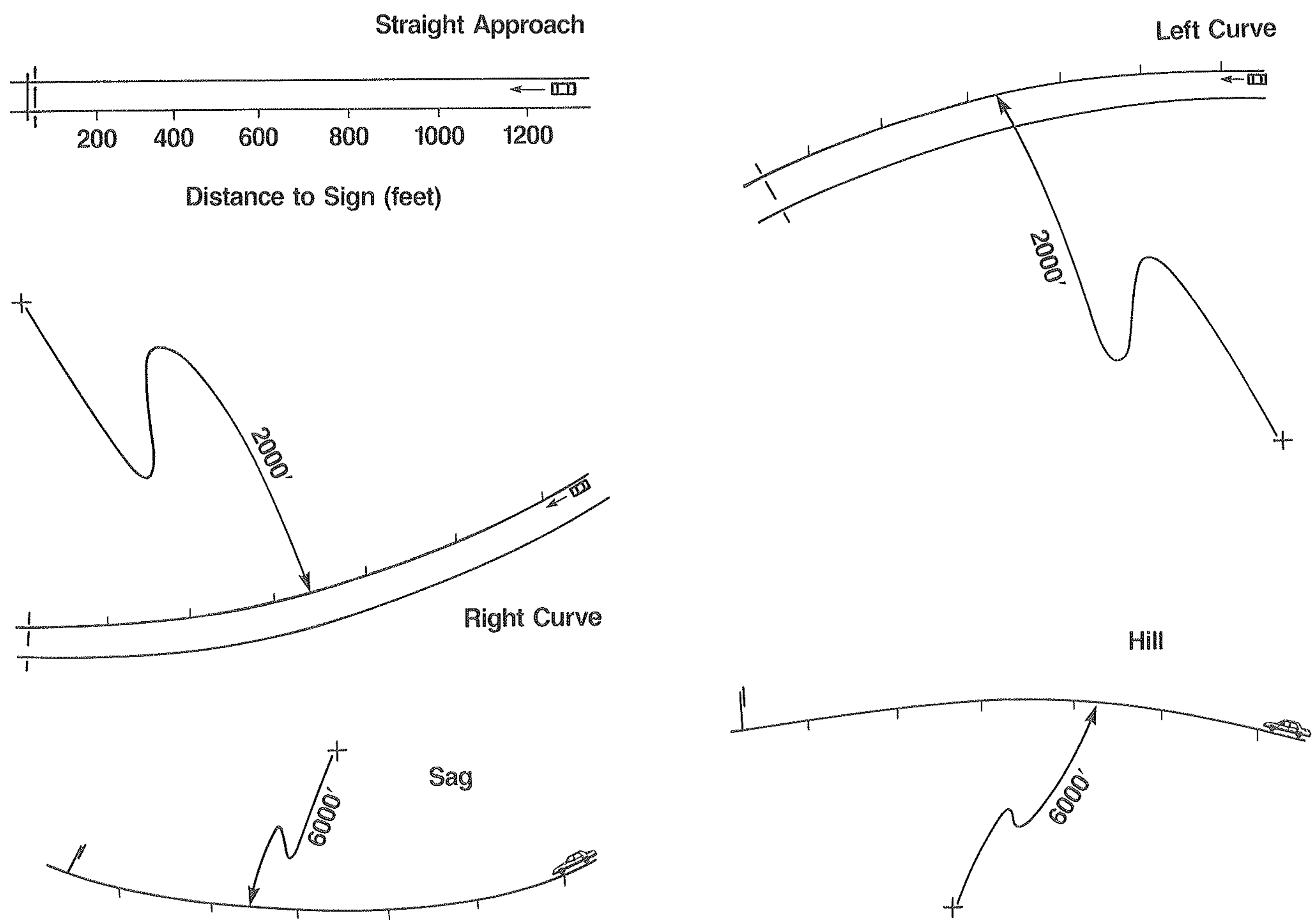


**FIGURE 2** **Roadway geometries.**

TABLE 1 RETROREFLECTIVE SIGN LUMINANCE, U.S. LOW-BEAM LAMPS—STRAIGHT TANGENT ROAD

| | Distance -- meters/feet | | | | | |
|---|---|---|---|---|---|---|
| Sign Mounting | 366/1200 | 305/1000 | 244/800 | 183/600 | 122/400 | 61/200 |
| | 7.32 | 9.53 | 12.15 | 14.67 | 15.58 | 9.53 |
| Right | 18.17 | 24.16 | 31.59 | 38.60 | 36.31 | 7.90 |
| | 52.42 | 72.07 | 94.74 | 119.43 | 101.76 | 15.75 |
| | 1.80 | 2.16 | 2.62 | 3.33 | 4.15 | 4.68 |
| Left | 4.49 | 5.49 | 6.79 | 8.87 | 9.83 | 7.86 |
| | 12.83 | 15.63 | 18.80 | 24.45 | 33.53 | 16.99 |
| | 1.97 | 2.21 | 2.44 | 2.52 | 3.07 | 2.21 |
| Overhead | 4.91 | 5.62 | 6.34 | 6.53 | 6.85 | 1.83 |
| | 14.22 | 16.62 | 18.78 | 20.28 | 19.76 | 3.12 |

Materials: (White) Enclosed Lens
Encapsulated Lens
Micro-prism

Luminance in Cd/m2

TABLE 2 RETROREFLECTIVE SIGN LUMINANCE, U.S. LOW-BEAM LAMPS—RIGHT CURVE

| | Distance -- meters/feet | | | | | |
|---|---|---|---|---|---|---|
| Sign Mounting | 366/1200 | 305/1000 | 244/800 | 183/600 | 122/400 | 61/200 |
| | O.18 | 1.04 | 1.74 | 2.64 | 5.71 | 9.53 |
| Right | 0.46 | 2.68 | 4.60 | 7.34 | 14.39 | 7.90 |
| | 1.16 | 9.24 | 17.14 | 23.94 | 33.17 | 15.75 |
| | 0.82 | 1.34 | 2.61 | 7.25 | 14.74 | 5.13 |
| Left | 2.17 | 3.58 | 7.16 | 20.42 | 35.42 | 9.01 |
| | 4.25 | 8.10 | 20.13 | 62.87 | 113.11 | 17.96 |
| | 0.53 | 0.85 | 1.14 | 1.89 | 2.63 | 2.33 |
| Overhead | 1.41 | 2.26 | 3.10 | 5.34 | 6.31 | 1.79 |
| | 2.83 | 5.81 | 10.75 | 18.07 | 15.71 | 3.11 |

Materials: (White) Enclosed Lens
Encapsulated Lens
Micro-prism

Luminance in Cd/m2

TABLE 3 RETROREFLECTIVE SIGN LUMINANCE, U.S. LOW-BEAM LAMPS—LEFT CURVE

| Sign Mounting | Distance -- meters/feet | | | | | |
|---|---|---|---|---|---|---|
| | 366/1200 | 305/1000 | 244/800 | 183/600 | 122/400 | 61/200 |
| | 0.64 | 1.01 | 1.68 | 2.78 | 5.43 | 9.53 |
| Right | 1.74 | 2.71 | 4.36 | 6.80 | 12.33 | 7.90 |
| | 3.53 | 6.85 | 12.59 | 20.87 | 40.71 | 15.75 |
| | 0.66 | 0.97 | 1.47 | 2.24 | 4.05 | 3.74 |
| Left | 1.79 | 2.60 | 3.77 | 5.26 | 8.82 | 5.47 |
| | 3.54 | 6.43 | 11.16 | 16.66 | 29.73 | 10.82 |
| | 0.40 | 0.64 | 0.84 | 1.17 | 1.76 | 2.06 |
| Overhead | 1.10 | 1.73 | 2.21 | 2.86 | 3.84 | 1.79 |
| | 2.06 | 3.69 | 6.01 | 9.64 | 12.62 | 3.82 |

Materials: (White) Enclosed Lens, Encapsulated Lens, Micro-prism

Luminance in Cd/m2

TABLE 4 RETROREFLECTIVE SIGN LUMINANCE, U.S. LOW-BEAM LAMPS—SAG

| Sign Mounting | Distance -- meters/feet | | | | | |
|---|---|---|---|---|---|---|
| | 366/1200 | 305/1000 | 244/800 | 183/600 | 122/400 | 61/200 |
| | * | 0.28 | 0.63 | 1.60 | 4.22 | 9.53 |
| Right | * | 0.71 | 1.66 | 4.25 | 9.66 | 7.90 |
| | * | 2.29 | 5.31 | 13.21 | 25.53 | 15.75 |
| | * | 0.37 | 0.60 | 1.00 | 1.84 | 3.09 |
| Left | * | 0.96 | 1.59 | 2.54 | 4.04 | 4.48 |
| | * | 2.96 | 4.80 | 7.93 | 12.79 | 9.87 |
| | * | 0.36 | 0.59 | 0.99 | 1.55 | 1.62 |
| Overhead | * | 0.95 | 1.57 | 2.58 | 3.21 | 1.20 |
| | * | 2.99 | 4.90 | 7.84 | 8.53 | 2.08 |

*Range exceeds headlamp field

Materials: (White) Enclosed Lens, Encapsulated Lens, Micro-prism

Luminance in Cd/m2

TABLE 5 RETROREFLECTIVE SIGN LUMINANCE, U.S. LOW-BEAM LAMPS—HILL

| Sign Mounting | Distance -- meters/feet | | | | | |
|---|---|---|---|---|---|---|
| | 366/1200 | 305/1000 | 244/800 | 183/600 | 122/400 | 61/200 |
| | * | * | * | 148.29 | 225.81 | 9.53 |
| Right | * | * | * | 386.61 | 526.84 | 7.90 |
| | * | * | * | 1130.50 | 1548.40 | 15.75 |
| | * | * | * | 28.33 | 25.60 | 9.11 |
| Left | * | * | * | 70.79 | 55.73 | 17.32 |
| | * | * | * | 187.54 | 198.71 | 37.65 |
| | * | * | * | 43.87 | 12.78 | 3.39 |
| Overhead | * | * | * | 112.64 | 30.48 | 3.14 |
| | * | * | * | 340.86 | 97.16 | 5.84 |

*Sign obscured by hill

Materials: (White) Enclosed Lens, Encapsulated Lens, Micro-prism

Luminance in Cd/m2

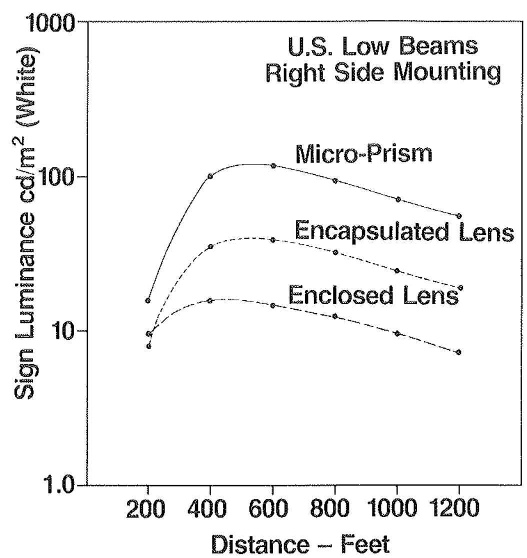


**FIGURE 3 Sign luminance—three retroreflective materials, straight tangent road.**

in a standard size passenger vehicle and are intended to provide a number of scenarios in which various approaches can be studied. The tables also permit comparisons for three differing sign positions, and two horizontal and vertical curves. Thus various retroreflective material types can be reviewed for a specific approach condition or distance and the appropriate type chosen that satisfies a given luminance criterion.

Figure 3 shows the performance difference for the three retroreflective materials. Figure 4 shows the effect of positioning the sign overhead or left relative to the right side. The effect of road curvature is given in Figures 5 and 6. As explained earlier, the headlamp matrix did not permit calculation of smaller radius curves, but, as can be visualized from the figures, sign luminance will be further impaired as compared to the data shown.

Tables 6 and 7 give sign luminance values for shoulder-mounted signs facing 30° away from and toward the roadway. A substantial number of these signs, some of which are highly critical, have been found at this entrance angle or greater. The luminance estimates of Tables 6 and 7 are derived from photometrically determined retroreflectance values appropriate for these entrance and observation angles.

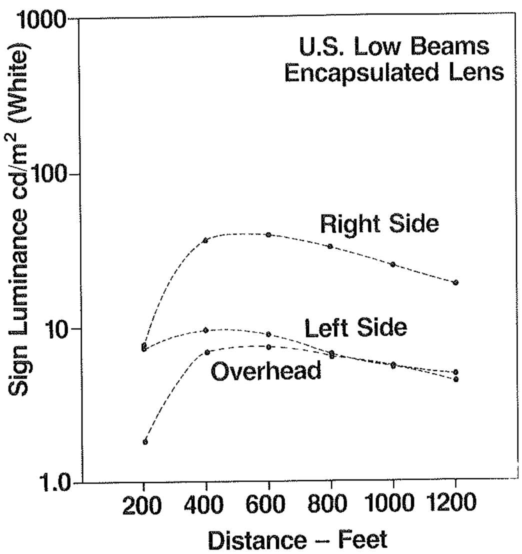


**FIGURE 4** **Sign luminance—three sign positions, straight tangent road.**

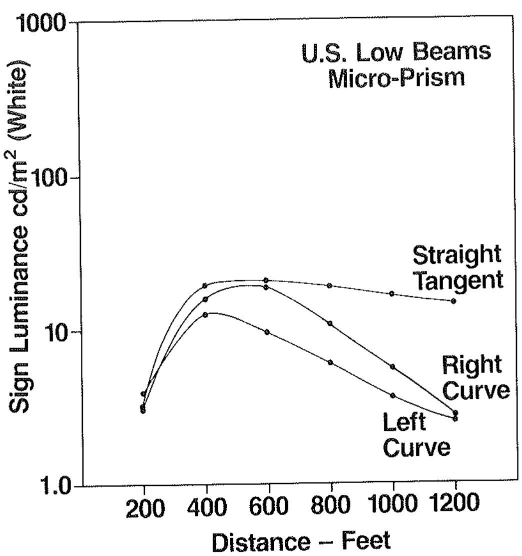


**FIGURE 5** **Overhead sign luminance—straight tangent versus right and left curves.**

## CONCLUSIONS

The use of luminance instead of retroreflectance for evaluating sign performance provides a means to match materials to roadway situations and driver needs. The driver's luminance needs must account for the variability of the driver population with a suitable factor of safety to accommodate driver age, expectancy, the complexity of the surround, and the criticality of the sign. These important factors are beyond the scope of this investigation. The luminance supply may be estimated from the table that offers the closest match to the

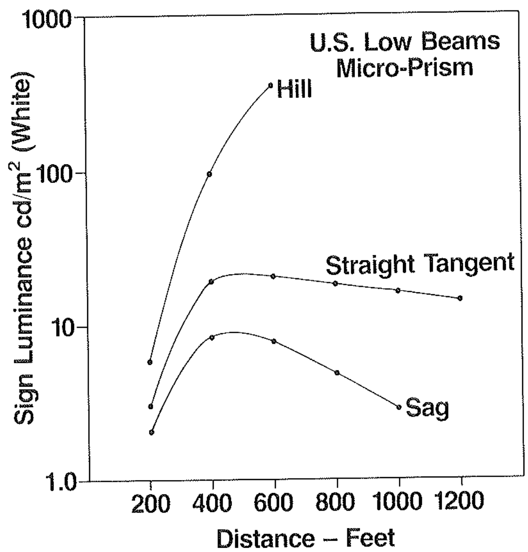


**FIGURE 6** **Overhead sign luminance—straight versus hill and sag.**

TABLE 6 RETROREFLECTIVE SIGN LUMINANCE, U.S. LOW-BEAM LAMPS—STRAIGHT TANGENT ROAD SIGNS ROTATED +30°

| | Distance -- meters/feet | | | | | |
|---|---|---|---|---|---|---|
| Sign Mounting | 366/1200 | 305/1000 | 244/800 | 183/600 | 122/400 | 61/200 |
| | 4.38 | 5.66 | 7.14 | 8.47 | 8.69 | 4.76 |
| Right | 13.01 | 17.21 | 22.33 | 26.92 | 24.58 | 4.93 |
| (Away) | 29.84 | 39.46 | 51.24 | 64.58 | 58.66 | 11.12 |
| | 1.18 | 1.43 | 1.77 | 2.30 | 3.28 | 3.94 |
| Left | 3.42 | 4.21 | 5.26 | 6.59 | 7.89 | 6.99 |
| (Toward) | 7.69 | 9.26 | 11.57 | 15.70 | 17.51 | 11.94 |

Materials: (White) Enclosed Lens
Encapsulated Lens
Micro-prism

Luminance in Cd/m2

TABLE 7 RETROREFLECTIVE SIGN LUMINANCE, U.S. LOW-BEAM LAMPS—STRAIGHT TANGENT ROAD SIGNS ROTATED −30°

| | Distance -- meters/feet | | | | | |
|---|---|---|---|---|---|---|
| Sign Mounting | 366/1200 | 305/1000 | 244/800 | 183/600 | 122/400 | 61/200 |
| | 4.71 | 6.17 | 7.94 | 9.80 | 10.81 | 7.37 |
| Right | 13.65 | 18.24 | 24.03 | 29.73 | 28.68 | 6.66 |
| (Toward) | 31.18 | 41.47 | 54.41 | 70.34 | 61.43 | 8.95 |
| | 1.06 | 1.25 | 1.50 | 1.85 | 2.37 | 2.03 |
| Left | 3.17 | 3.84 | 4.70 | 5.68 | 6.42 | 4.48 |
| (Away) | 7.20 | 8.64 | 10.65 | 13.83 | 15.75 | 12.51 |

Materials: (White) Enclosed Lens
Encapsulated Lens
Micro-prism

Luminance in Cd/m2

roadway situation. Practitioners may then select the materials that offer the luminance level desired through the range of distances that may be of interest. In this manner, material selection can become a part of the design process.

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# Effects of Light Sources on Highway Sign Color Recognition

Syed F. Hussain, John B. Arens, and Peter S. Parsonson

**A wide variety of light sources is used for externally illuminated highway signs. Some of these light sources change the color appearance of signs at night. This study evaluates acceptable alternative light sources for illuminating highway signs. Light sources investigated included incandescent, fluorescent, metal halide, mercury, high pressure sodium, and low pressure sodium lamps. The metal halide lamp performed best overall and is recommended to illuminate a broad range of highway sign colors. This also could include the use of metal halide lamps in future automobile headlights. Mercury lamps that are economical and provide good color rendition on green, blue, and white are recommended for overhead signs. With some compromise on the color rendition, high pressure sodium is another cost-saving alternative for overhead signs. High pressure sodium is also the best choice to illuminate construction and maintenance signs.**

Highway signs are designed to provide information, guidance, and regulations for the safe and efficient flow of traffic. Traffic signs essentially contain shape, color, and legend. Shape and color are important because they convey the meanings and messages of signs before the legend can be read. Colors on signs convey certain meanings and significance to road users. Regulatory signs, usually with white or red backgrounds, are used for traffic laws and regulations. Yellow is used as the background color for warning signs; orange for construction zone signs; and green, brown, and blue are used for guidance, services, and information, respectively. Road users expect to see these color codes for their assigned meanings. Therefore, the appearance of these colors should remain the same during nighttime as during daylight. For nighttime recognition, signs can be placed generally into two categories: (a) Externally (and in some cases internally) illuminated signs and (b) non-illuminated (generally retroreflective) signs. The *Manual on Uniform Traffic Control Devices* (*1*) requires that highway signs intended to be used during the hours of darkness shall be either retroreflective or illuminated to show approximately the same shape and color day and night.

Recognition of non-illuminated signs depends on the amount of light from headlights falling on the sign. Legibility and recognition of externally illuminated signs depends on the type and amount of fixed illumination. Therefore, the selection of light sources to illuminate overhead guide signs is very important. Motorists may endanger themselves and others by slowing down or stopping because signs may not be legible and identifiable at distances necessary to take action. Proper illumination of highway signs is critical for the accurate and timely recognition of sign colors and messages. Selection of adequate illumination and the appropriate type of illuminants becomes even more important when:

- Ambient luminance/background complexities increase,
- Volume of traffic increases,
- Complexity of the highway design increases, or
- Adverse weather conditions prevail at certain locations.

The design of a highway lighting system is governed by lamp efficacy, optical controllability, color rendering properties, lamp life, and initial as well as operational costs. In recent years, energy constraints and economics have played major roles in selecting sign lighting systems. The importance of other factors has become somewhat reduced. This trend has led to selecting some light sources that distort the color appearance of signs at night.

Color appearance of an object is a function of the light source illuminating it and the spectral selectivity of the object itself. Colors of highway signs, when illuminated by different light sources, may not look the same at night as during the day. Knowledge of changes in the color appearance of signs is important because:

- A wide variety of light sources are available and used by various agencies responsible for sign lighting.
- Light sources for automobile headlamps may change, replacing the conventional incandescent or halogen lamps with high intensity discharge lamps such as metal halide lamps.
- The potential exists for changing sign materials by bringing them closer to CIE (Commission Internationale de l'Eclairage) colors.

## OBJECTIVE

The objective of this study was to develop guidelines to select efficient and cost effective light sources to illuminate highway signs. The recommendations are based on the color rendering characteristics of the light sources as well as their ability to meet the minimum illumination levels recommended by AASHTO.

S. F. Hussain, Bellomo-McGee, Inc., 8330 Boone Boulevard, Suite 700, Vienna, Va. 22180. J. B. Arens, FHWA, 6300 Georgetown Pike, McLean, Va. 22101. P. S. Parsonson, Georgia Institute of Technology, Atlanta, Ga. 30332.

## BACKGROUND

Many of the safety colors used for denoting hazardous situations in industrial applications are not identified accurately under common high intensity discharge (HID) light sources (*2,3*). This is equally true for highway sign colors when illuminated under these lamps.

Until the early 1950s, incandescent lamps were the most widely used light sources for street lighting and sign illumination. Good color rendering properties, simple installation, low initial costs, and easy maintenance were some of their major advantages. Short service life and low luminous efficacy were their main disadvantages. Today, the use of this lamp for roadway lighting is almost phased out.

As traffic signs grew in size, fluorescent lamps were used extensively. These lamps illuminate signs uniformly and provide very good color rendition. But they are cumbersome to maintain and susceptible to temperature variations (resulting in considerably reduced light output and starting problems during cold weather). Because of their large size (resulting in relatively large, heavy, and expensive luminaires), they were never used extensively for street lighting, but are still used for sign lighting by some states. In the mid-1960s, mercury lamps were introduced to illuminate signs. They are long-lasting and need very little maintenance. The color rendition of clear mercury lamps, however, is somewhat poor. Phosphor-coated (color-improved) mercury lamps, which provide better color rendition, have been added to the list of widely used sign lighting sources. Because of their many advantages, mercury lamps are considered a very good choice for sign lighting and are used widely.

Metal halide lamps, similar in operation, construction, and performance to mercury lamps, but with considerably better color rendition, were introduced in the late 1960s. Although metal halide lamps have very good color rendition, they have never become a favorite for sign lighting because they have a shorter life span and higher cost than mercury lamps. However, some cities and states do use this light source as a sign illuminant because of its excellent color rendition.

The energy crisis of the early 1970s gave a sudden and dramatic boost to the use of high pressure sodium lamps. There was a wholesale change to high pressure sodium lamps for street lighting and quite a few cities and states also applied this light source to sign lighting. While this lamp has very high luminous efficacy, it distorts the colors of signs considerably. Although low pressure sodium lamps for sign lighting are not suggested, they are included in this study. Low pressure sodium lamps have limited application for sign lighting, but their use for street lighting in some areas along the West Coast, especially where astronomical observatories are located, is becoming more common. Although this is the most efficient light source, all of the light produced is monochromatic yellow (i.e., all the energy emitted is contained in a very narrow band within the visible spectrum). This distorts colors completely; only yellow and orange are distinguishable.

## STUDY APPROACH

A test apparatus was built to determine the percentage of people that might be confused by various light sources if such lighting were the only source of sign lighting. The test apparatus consisted of various light sources, an array of highway sign color samples, a microcomputer, and two electric motors. The computer was used for data recording and operating the electric motors to generate random movements of the color samples and the light sources. The light sources were contained in a ventilated box with openings at the bottom. Partitions between the individual lamps prevented light from one source from becoming mixed with the light emitted by adjacent lamps. The light box rotated in the horizontal plane above the viewing box until one of the openings of the light box matched up with the opening in the top of the viewing box. When these two openings were coincident, the light from the lamp being evaluated was evenly reflected to the color sample at the back of the viewing box by a 45-degree inclined diffused reflector. This reflector was made from a white material covered with a layer of non-selective white blotter paper. To provide uniform and approximately equal levels of illuminance, the openings through the lamp compartment were adjusted with perforated screens containing holes of different diameters. Table 1 gives illuminance data of the various light sources with the illuminance meter held at the sample locations before the openings of the lamp compartments were adjusted. Table 2 presents luminance data measured from white blotter paper held at the sample position with the light flux attenuators installed.

Externally illuminated signs are illuminated either with the luminaire positioned above and light falling at an angle to the surface of the sign or the luminaire is installed at the bottom of the sign structure, illuminating it at an angle from the bottom to the face of the sign. The test apparatus was built to simulate a situation where the lighting fixture is installed above the sign. Color samples were attached to a circular wheel made of 2 mm aluminum that rotated in a vertical plane at the back of the viewing box. There was a 4-in. × 4-in. opening at the rear wall of the viewing box through which one color sample, brought in place by selective rotation of the color wheel, could be observed. Another opening of 1½ in. × 4 in. was provided for the observer, who was located at the other end of the viewing box. Two baffles were installed in the viewing box to restrict the observer's view to exactly the size of the sample to avoid possible glare on the surface of the sample. A manually operated shutter at the observer's end of the viewing box was used to prevent the observer from seeing the color samples as they and the light sources were moved between observations. This was done to avoid possible bias of the observer due to light source and color sample movements. All observations were taken in a dark room, and only one light source illuminated one randomly selected color sample at a time.

The sequence of selection of light source/color combination was changed for each subject to eliminate any possible ordering effect. Exposure of each color sample was brief, yet long enough to allow the observers time to identify each one. During the experiment, the samples were viewed only once, except for a few repetitions to assess the observer's consistency of recognition.

During the first part, each color sample, one at a time, was illuminated with one of the light sources contained in the light box and viewed by the subject. A total of 63 different color sample/light source combinations were exposed to each sub-

TABLE 1 ILLUMINANCE DATA AND COLOR COORDINATES OF LIGHT SOURCES MEASURED AT THE SAMPLE POSITION BEFORE ADJUSTMENT

| Light Source | Illuminance Lux | x | y |
|---|---|---|---|
| Incandescent | 93.90 | .457 | .403 |
| Fluorescent | 89.00 | .371 | .380 |
| Clear MH | 142.00 | .349 | .379 |
| Color Imp. MH | 97.20 | .382 | .385 |
| Clear Mercury | 126.00 | .308 | .387 |
| Color Imp. Mercury | 112.00 | .366 | .391 |
| Clear HPS | 118.00 | .523 | .424 |
| Color Imp. HPS | 64.40 | .516 | .416 |
| LPS | 88.90 | .557 | .440 |

ject. Observers were asked to name each of the colors they saw. In the second part, the subjects viewed a combination of miniature traffic signs, representing real-world highway signs. The signs used included a STOP sign, an overhead guide sign with green background, and a NO OUTLET warning sign. These signs cover a broad range of important colors used for roadway signs. The signs were illuminated with one light source first and then with another. Subjects were asked to indicate their preference. The paired comparison approach was used to rate the most preferred light source. The comparison was done on a "knock down" basis, i.e., after comparing four pairs in the first step, only four light sources were included in the next step. Finally, only three light sources were used in the third step to select the most preferred lamp.

## SUBJECTS

The Highway Research Center at Turner-Fairbank maintains a subject pool from the driving public who have either participated in previous studies in the facility or have indicated their willingness to participate in upcoming studies. Initially, 40 subjects were selected, 20 male and 20 female, evenly divided into four groups of under 25, 26–40, 41–54, and over 55 years of age. Because of some mechanical problems with the apparatus as well as time constraints, the first part of the study is based on the response from 33 subjects; the second part contains data from 43 subjects. The overall range of subject age was from 18 to 67 with an average of 40.

All subjects were licensed drivers. Their visual acuity was tested to verify a threshold level of 20/40. Subjects were given a color vision test using pseudoisochromatic plates to identify any color defectiveness. Three subjects from the male groups indicated color defectiveness. The color defective drivers were included in the study to assess their ability to recognize sign colors.

## GENERAL FINDINGS

Fluorescent, mercury, high pressure sodium, and metal halide are the most commonly used light sources for highway sign

TABLE 2 LUMINANCE DATA AND COLOR COORDINATES MEASURED FROM BLOTTER PAPER AT THE SAMPLE POSITION

| Light Source | Luminance cd/m2 | x | y |
|---|---|---|---|
| Incandescent | 33.70 | .467 | .404 |
| Fluorescent | 38.10 | .378 | .384 |
| Clear MH | 42.60 | .358 | .384 |
| Color Improved MH | 33.30 | .390 | .388 |
| Clear Mercury | 45.00 | .311 | .389 |
| Deluxe Mercury | 40.90 | .371 | .392 |
| Clear HPS | 37.20 | .537 | .415 |
| Color Imp. HPS | 23.80 | .525 | .415 |
| LPS | 34.40 | .576 | .423 |

NOTE: Lamp compartment openings adjusted to provide approximately equal illumination.

lighting. Incandescent and low pressure sodium lamps have very limited applications for sign lighting. However, these lamps were included in the experiment because incandescent lamps have been used extensively for sign lighting in the past and the low pressure sodium lamp might be a future candidate for this purpose. The correct identification of various colors under different lamps is given below:

*Under mercury vapor lamp:*

| | |
|---|---|
| RED SAMPLE | appeared RED to 9% of subjects.<br>appeared PINK to 24% of subjects. |
| YELLOW SAMPLE | appeared YELLOW to 18% of subjects.<br>appeared GREEN to 55% of subjects. |
| ORANGE SAMPLE | appeared ORANGE to 6% of subjects.<br>appeared YELLOW to 39% of subjects. |
| BROWN SAMPLE | appeared BROWN to 9% of subjects.<br>appeared GRAY to 27% of subjects. |

*Under metal halide lamp:*

| | |
|---|---|
| RED SAMPLE | appeared RED to 58% of subjects.<br>appeared ORANGE to 27% of subjects. |
| YELLOW SAMPLE | appeared YELLOW to 73% of subjects.<br>appeared OLIVE to 12% of subjects. |
| ORANGE SAMPLE | appeared ORANGE to 21% of subjects.<br>appeared PEACH to 30% of subjects. |
| BROWN SAMPLE | appeared BROWN to 21% of subjects.<br>appeared TAN to 39% of subjects. |

*Under high pressure sodium lamp:*

| | |
|---|---|
| RED SAMPLE | appeared RED to 33% of subjects.<br>appeared ORANGE to 52% of subjects. |
| YELLOW SAMPLE | appeared YELLOW to 45% of subjects.<br>appeared ORANGE to 52% of subjects. |
| WHITE SAMPLE | appeared WHITE to 3% of subjects.<br>appeared ORANGE to 42% of subjects. |
| GREEN SAMPLE | appeared GREEN to 30% of subjects.<br>appeared BLUE to 30% of subjects. |
| BLUE SAMPLE | appeared BLUE to 24% of subjects.<br>appeared GRAY to 36% of subjects. |
| BROWN SAMPLE | appeared BROWN to 27% of subjects.<br>appeared ORANGE to 30% of subjects. |

*Under fluorescent lamp:*

No real confusion occurred under this lamp except for BROWN.

| | |
|---|---|
| BROWN SAMPLE | appeared BROWN to 36% of subjects.<br>appeared TAN to 30% of subjects. |

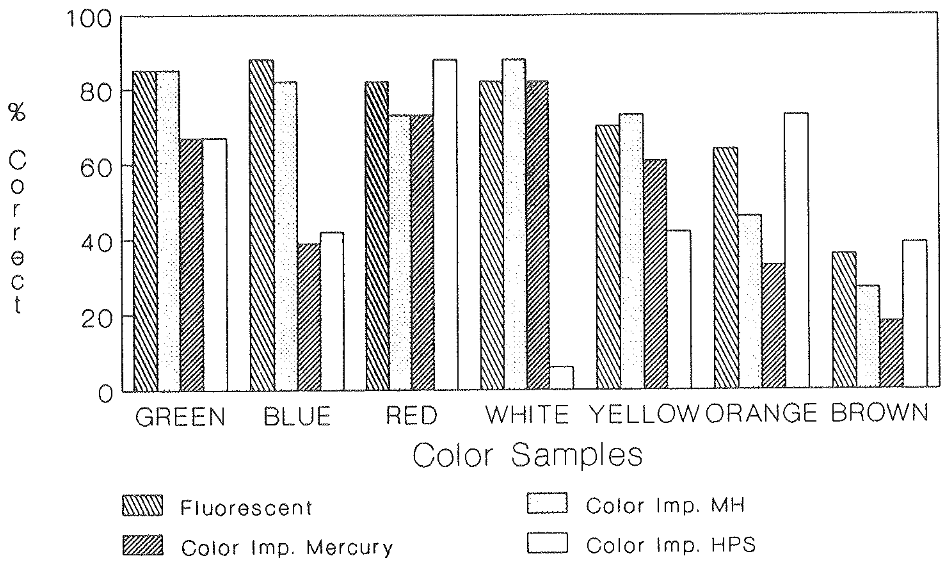


**FIGURE 1 Correct identification of different colors under various lamps.**

The preceding table indicates a striking difference in performance of the different light sources for illuminating various sign colors. The low pressure sodium lamp performed very poorly; red, green, and blue were not correctly identified with this lamp. The fluorescent lamp appeared to perform substantially better when compared with the other light sources. The difference is not significant, however, when compared with the incandescent or the color-improved metal halide lamps. Figure 1 shows the correct identification of various colors under four light sources commonly used for sign lighting. A quick look at the data above shows that highway brown was rarely identified as brown under any light source. Brown was mingled mostly with tan and cream; it was confused with orange or yellow under low pressure sodium. Under mercury lamps, yellow was confused with green and red with orange. Recognition of all colors except red was improved significantly for metal halide lamps. Red was confused with orange and orange with yellow. It is noteworthy that highway yellow was confused with construction zone orange under almost all light sources. Yellow, used for warning signs, appears close to construction zone orange. A current FHWA research study is investigating this problem.

It should be noted that different light sources affect the appearance of colors to varying degrees. The major factor affecting color appearance is the spectral composition of the light emitted by the source. Visual radiations (light) are contained in a region of the electromagnetic spectrum between 400 and 700 nm. Just below 400 nm are invisible ultraviolet radiations and above 700 nm are infrared radiations. Each light source emits a different composition of spectral energy. A basic knowledge of how this affects color rendering of objects and a knowledge of the spectral energy distribution of various light sources are necessary to understand the problem investigated in this study.

Figures 2 through 7 show the spectral energy distributions of the lamps used in this study. Absence or overabundance of a particular wavelength in a light source may cause serious distortion of a particular color when illuminated by such a light source. The low pressure sodium lamp, for example, emits monochromatic yellow energy (single wavelength only). As a result, the only colors that can be recognized correctly under this light are yellow and orange. All other colors are distorted. Fluorescent and metal halide lamps contain spectral energy from all ranges of the spectrum and so provide very good color rendition for most colors.

## STATISTICAL ANALYSIS

An appropriate statistical test is determined by the nature and level of the measurement for a particular set of data under consideration. Another important factor is knowledge of the distribution of the sampling population. In the case of no previous knowledge of the population distribution, nonparametric tests are the best choice. Data collected in this experiment were both nominal and dichotomized as "yes" or "no." Therefore, without any assumption of the sampling population, a Cochran $Q$-test was executed to determine whether the performance of each light source based on subject responses was statistically different for illumination of various sign colors. Table 3 demonstrates that each light source exhibited significantly different performance for the various sign colors except brown. That is, no matter what light source is used for the brown color sample, the appearance of this color may still be confused.

Chi-square comparisons of various light sources were made to rate them on their overall performance. The data given in Table 4 demonstrate that the performance of the fluorescent lamp is significantly better when compared to clear mercury and to both clear and color-improved high pressure sodium (HPS) lamps. The fluorescent lamp did not perform significantly better than incandescent, metal halide (both clear and color-improved), and color-improved mercury lamps. The incandescent lamp performed significantly better than the clear

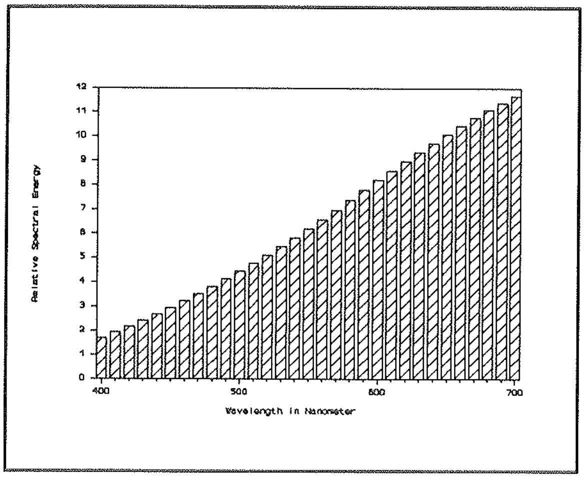


**FIGURE 2** Spectral energy distribution of incandescent lamp.

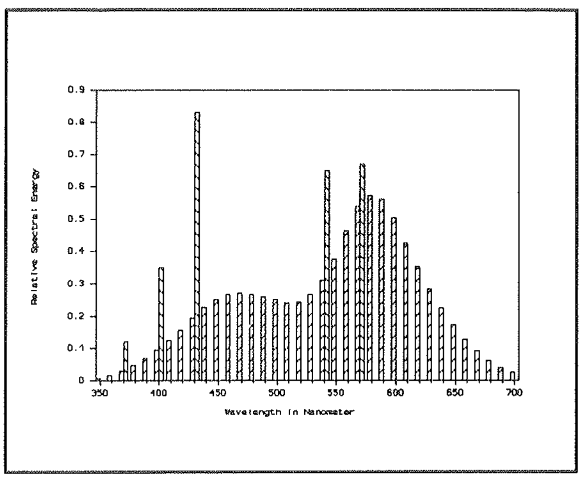


**FIGURE 3** Spectral energy distribution of fluorescent lamp.

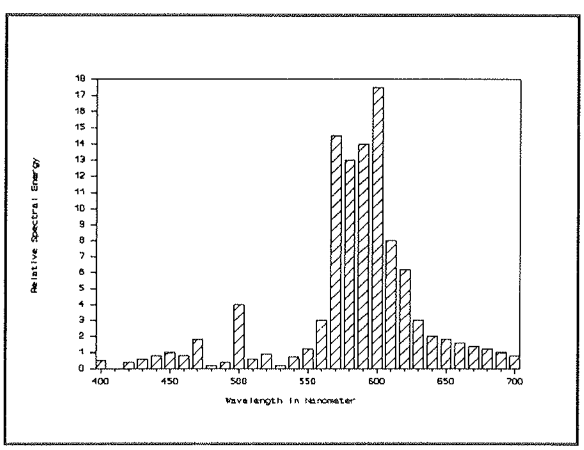


**FIGURE 4** Spectral energy distribution of HPS lamp.

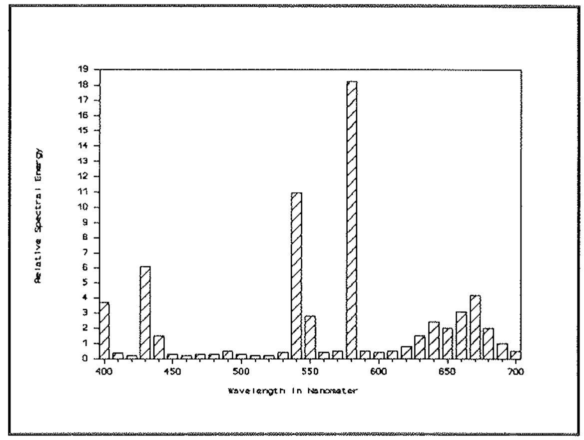


**FIGURE 5** Spectral energy distribution of mercury lamp.

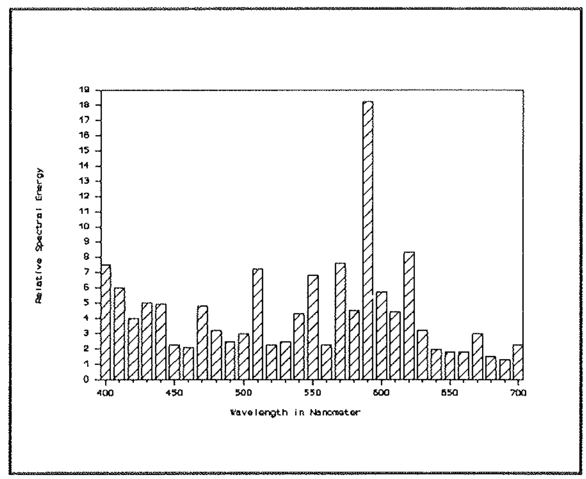


**FIGURE 6** Spectral energy distribution of metal halide lamp.

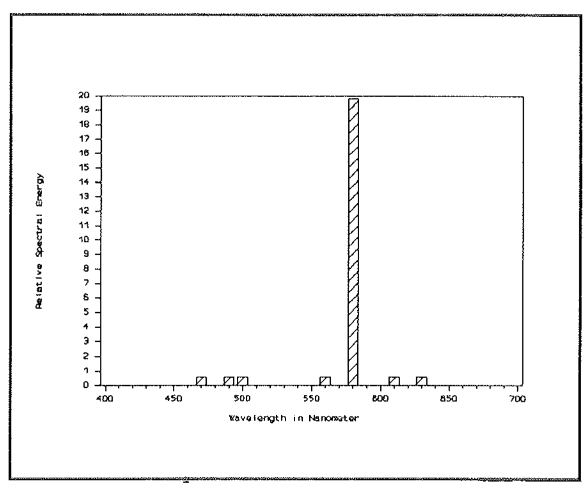


**FIGURE 7** Spectral energy distribution of LPS lamp.

TABLE 3 COCHRAN $Q$-TEST RESULTS FOR VARIOUS COLORS

| Color | Q-Value | DF | Probability |
|---|---|---|---|
| White | 146.7 | 6 | <.001 |
| Red | 145.9 | 6 | <.001 |
| Blue | 126.5 | 6 | <.001 |
| Green | 105.2 | 6 | <.001 |
| Orange | 78.5 | 6 | <.001 |
| Yellow | 38.5 | 6 | <.001 |
| Brown | 15.5 | 6 | <.05 |

metal halide and both versions of the mercury and HPS lamps, but did not perform significantly different from the color-improved metal halide lamp. Both the clear and the color-improved versions of the metal halide lamps performed significantly better than the clear mercury and both versions of the HPS lamps. Mercury lamps (clear and color-improved) exhibited better performance than either clear or color-improved HPS lamps. Performance between clear and color-improved HPS lamps was not statistically different.

In the second part of the study, subjects viewed several scaled-down signs against a dark background. Each subject saw these signs illuminated alternatively by a pair of relatively similar light sources, i.e., color-improved high pressure sodium and clear high pressure sodium; color-improved mercury and clear mercury; and color-improved metal halide and clear metal halide. Subjects expressed a preference for one light source for each pair viewed. The preferred light sources were used to again illuminate the signs and again the subjects expressed preferences based on overall appearance and not necessarily on true color recognition.

In comparing the two types of high pressure sodium in the first group, 77 percent of the subjects preferred the color-improved lamp. Eighty-four percent of the subjects preferred the color-improved mercury lamp over the clear mercury lamp, and 58 percent preferred the color-improved metal halide lamp over the clear metal halide lamp.

Once the choice was narrowed down to the color-improved versions of the high pressure sodium, mercury, and metal halide lamps, the HPS lamp was compared to the mercury lamp and the metal halide lamp was compared to the fluorescent lamp. This "pre-selection" of pairs to compare may have led to a slightly biased result; however, it was felt that by grouping the clear and color-improved versions first and then the best two sets (metal halide versus fluorescent and HPS versus mercury), a true preference scaling could be developed. These short-cuts in the paired comparisons were necessitated by time constraints.

Results from the second step indicated that 91 percent of the observers preferred the mercury lamp over the HPS lamp and only 51 percent of the observers preferred the metal halide lamp over the fluorescent lamp. Because there was

TABLE 4 CHI-SQUARE COMPARISONS OF VARIOUS LIGHT SOURCES

| Light Sources Comparisons | | | Probability |
|---|---|---|---|
| Fluorescent | vs. | Incandescent | N.S. |
| Fluorescent | vs. | Imp MH | N.S. |
| Fluorescent | vs. | Clear MH | N.S. |
| Fluorescent | vs. | Imp Mercury | N.S. |
| Fluorescent | vs. | Clear Mercury | <.0003 |
| Fluorescent | vs. | Imp-HPS | <.0011 |
| Fluorescent | vs. | Clear HPS | <.0002 |
| Incandescent | vs. | Imp MH | N.S. |
| Incandescent | vs. | Clear MH | <.0143 |
| Incandescent | vs. | Imp Mercury | <.0415 |
| Incandescent | vs. | Clear Mercury | <.0000 |
| Incandescent | vs. | Imp HPS | <.0132 |
| Incandescent | vs. | Clear HPS | <.0003 |
| Imp MH | vs. | Clear MH | N.S. |
| Imp MH | vs. | Imp Mercury | N.S. |
| Imp MH | vs. | Clear Mercury | <.0028 |
| Imp MH | vs. | Imp HPS | <.0000 |
| Imp MH | vs. | Clear HPS | <.0000 |
| Clear MH | vs. | Imp Mercury | N.S. |
| Clear MH | vs. | Clear Mercury | <.0265 |
| Clear MH | vs. | Imp HPS | <.0000 |
| Clear MH | vs. | Clear HPS | <.0000 |
| Imp Mercury | vs. | Clear Mercury | <.0004 |
| Imp Mercury | vs. | Imp HPS | <.0000 |
| Imp Mercury | vs. | Clear HPS | <.0000 |
| Clear Mercury | vs. | Imp HPS | <.0000 |
| Clear Mercury | vs. | Clear HPS | <.0000 |
| Imp HPS | vs. | Clear HPS | N.S. |

little difference between the metal halide and the fluorescent lamps, the third step compared the mercury, metal halide, and the fluorescent lamps. The mercury lamp was preferred by 44 percent of the subjects, and 28 percent of the subjects preferred either the metal halide or fluorescent lamp.

## CONCLUSIONS AND RECOMMENDATIONS

Fluorescent, metal halide, mercury, and high pressure sodium lamps are widely used light sources for sign lighting. The results of this study indicate that fluorescent lamps provide slightly better color rendition/color recognition than the next best light source, the metal halide lamp. However, because of the higher purchase and operating costs of a fluorescent lamp system, along with starting problems and reduced light output during cold weather, metal halide lamps are a better choice. Therefore, where color identification is the most important criterion, metal halide lamps are recommended. This also could include the use of metal halide lamps in future automobile headlights.

For overhead lighting of signs with green backgrounds and white legends or blue backgrounds and white legends, savings can be achieved by using mercury lamps. Mercury lamps, especially the color-improved version, do not seriously distort these sign colors. The least desirable light source, from a color rendering standpoint, is the high pressure sodium lamp. However, even this lamp provides an acceptable color rendition where system and operational costs are more important than true color recognition. It is the opinion of the authors that a directional sign that can be seen and read, even if the colors are distorted to some degree, is considerably better than a sign which is dark, invisible, or illegible.

For construction and maintenance areas where orange signs are used and illuminated, high pressure sodium lamps are a very good choice. This lamp is economical and provides very good color rendition of orange signs.

Because of the extreme absence of any color recognition other than yellow or orange, the low pressure sodium lamp is not recommended for sign lighting. For economic reasons, the incandescent lamp (including any quartz-halogen lamps) also should not be considered for sign lighting except where the lighting is temporary and equipment costs are to be held to a minimum.

In the present experiment, nonretroreflective signing materials were used. It must be recognized that the real-world nighttime color appearance of signs is a function of both the fixed lighting and the vehicle's headlamps. Fixed lighting provides the visual component of the diffusing quality of the sign surface. Headlamps deliver the retroreflective component. We used diffuse color samples and diffuse lighting geometry in our experiment, which represents only the visual component. Our rationale was that unless you are relatively close to a lighted sign, or use high beams, most headlamps will have little impact on the color appearance of a sign. We understand the FHWA is considering another study using retroreflective color samples to investigate the impact of low beam headlamps, when added to fixed lighting, on the color appearance of a sign. It may also be interesting to include some very small incandescent light sources in the visual field to simulate color reference points such as automobile headlamps, taillights, or surrounding ambient lighting.

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# Conspicuity of Suprathreshold Reflective Targets in a Driver's Peripheral Visual Field at Night

HELMUT T. ZWAHLEN

**Past investigations and experimental studies dealing with the visual detection of either nonreflectorized or reflectorized objects or targets in the driving environment at night have been limited primarily to foveal or line-of-sight visual detection. A geometric model is developed to analyze reflectorized targets located ahead of a car at different locations along a tangent-curve and curve-tangent section of a highway. Typical night driving eye scanning data are also presented. The model demonstrates that in many cases unknown or unexpected reflectorized targets, such as a reflectorized license plate or an advance warning sign, will appear initially at moderately large peripheral angles up to 15° or more away from a driver's foveal eye fixation point, or line of sight. A field study involving the foveal and peripheral detection of a reflectorized target is presented to show that peripheral visual detection distances decrease considerably as the peripheral visual angle away from the fovea, or line of sight, increases. A 10° peripheral visual detection angle results in an average visual detection distance approximately one-half of the average foveal detection distance. It is concluded that in a situation where drivers approach or negotiate a curve at night, where reflectorized objects or targets will become visible for the first time probably in the periphery of a driver's visual field, and where there is a need for early detection, the reflectivity of the target should be increased to ensure timely recognition, information processing, and decision making, and appropriate control actions.**

Past investigations and experimental studies dealing with the visual detection of either nonreflectorized or reflectorized (made with reflective materials) targets in the driving environment at night have been limited primarily to foveal or line-of-sight visual detection. One exception in the current literature is a field study by Zwahlen (*1*) that investigated the ability of human subjects to detect an approaching reflectorized target at night in the field foveally and at peripheral visual angles of 10°, 20°, and 30°. Zwahlen found that at a 10-degree peripheral angle the average detection distance was 47 to 59 percent of the average foveal detection distance. At a 30-degree peripheral angle this distance declined to 25 to 33 percent of the average foveal detection distance. Past investigations of a driver's recognition capabilities also have been limited primarily to foveal or line-of-sight recognition of symbols or shapes of targets. One exception is a laboratory study by Karttunen and Hakkinen (*2*) that investigated the ability of subjects to recognize symbolic road signs commonly used in Finland at peripheral angles of 10°, 20°, 30°, 40°, and 50°. They found that when signs that subtended a visual angle of 4° of visual arc (from bottom to top) were projected on a screen in a laboratory for 125 ms, the ability of subjects to identify the road sign decreased from 100 percent for foveal presentation to 92.4 percent for a peripheral angle of 10° and to 32.5 percent for a peripheral angle of 50°.

Department of Industrial and Systems Engineering, Ohio University, Athens, Ohio 45701-2979.

Because Zwahlen (*1*) showed that a driver's ability to detect a reflectorized target at night decreases considerably as the peripheral angle at which the target is first presented increases and Karttunen and Hakkinen (*2*) showed similar results for peripheral recognition accuracy of commonly used symbolic road signs, data based solely on human foveal visual detection capabilities in the design of reflectorized targets in the highway environment may be inadequate if such a target is likely to first appear in the periphery of a driver's visual field and early detection is needed. Therefore, the objective of this paper is to investigate the importance of peripheral visual detection in the driving environment at night.

## MAGNITUDE OF PERIPHERAL VISUAL DETECTION ANGLES IN THE HIGHWAY ENVIRONMENT

Knowledge of human peripheral visual detection capabilities would not be useful in traffic safety if it were not possible to demonstrate that driving situations exist where targets are likely to first appear in a driver's field of view at relatively large peripheral visual detection angles. Since highways are designed on geometric principles, it was possible to develop a geometric model to determine the peripheral visual detection angles that exist for particular driving situations and for targets in the driving environment. A computer was used to analyze multiple situations where large peripheral visual detection angles might be found.

The geometric model was based on the assumptions that: (a) the driver looks ahead of the car in a direction that is parallel to the longitudinal center of the car, (b) the driver is driving in the center of the right-hand lane on a two-lane highway, (c) the driver is driving on a level and flat road surface as opposed to a road with vertical curves, and (d) there are no physical barriers, objects, or foliage along the highway that might obstruct a driver's direct view of the target of interest. The analytical model was developed to evaluate tangent-curve sections of a highway, where a driver

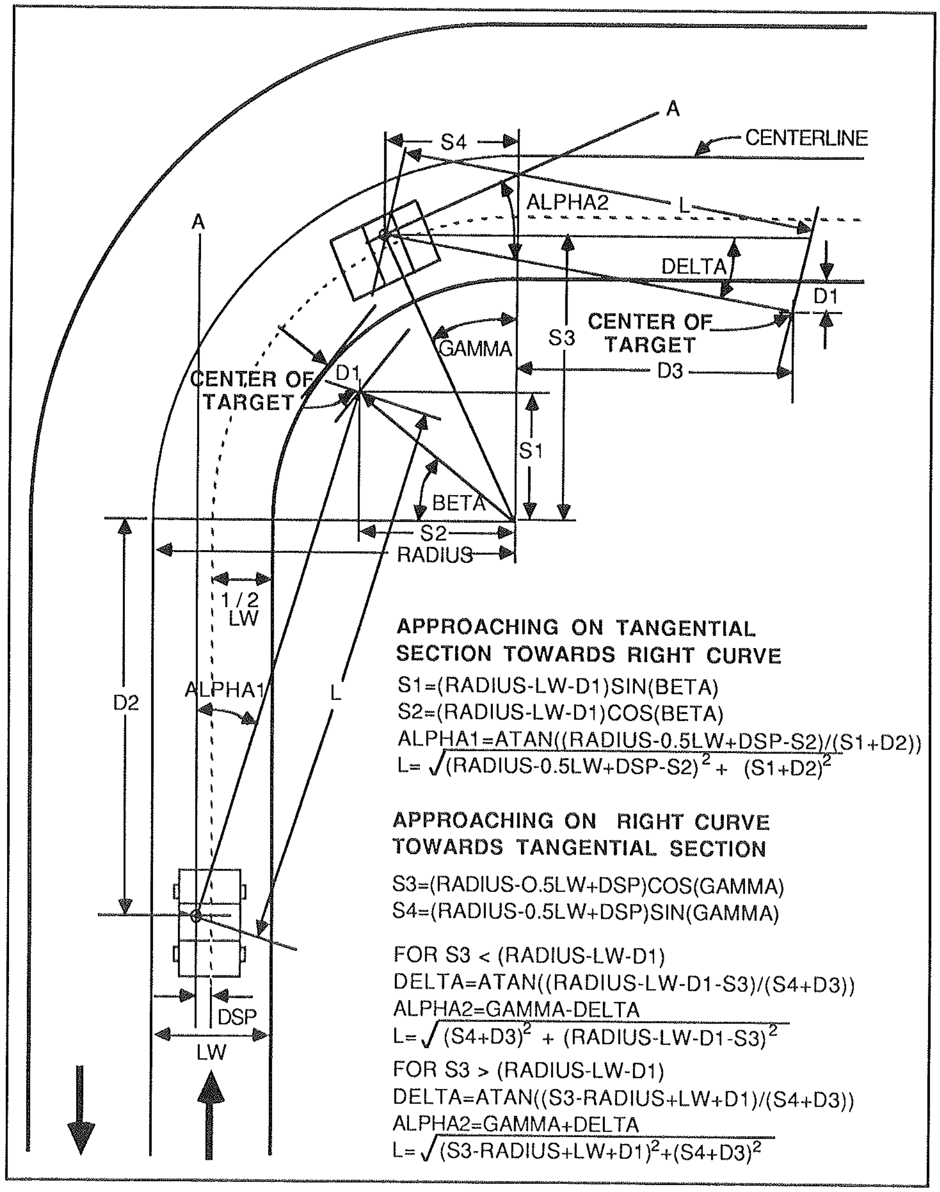


**FIGURE 1 Geometric configuration and equations to calculate the peripheral visual detection angles for tangent-curve and curve-tangent sections for a right curve of a two-lane highway.**

is on a tangent section of a highway appproaching a target located in the curve section of the highway, and curve-tangent sections of a highway, where a driver is negotiating a curve while approaching a target located along the tangent section of the highway beyond the end of the curve. Large peripheral visual detection angles may also occur when a driver is negotiating a long curve and is approaching a target located farther ahead in the same curve. However, this was not analyzed separately because when the driver and target are separated by short distances in the tangent-curve or curve-tangent sections of a highway both the driver and the target are very close to being in the same curve. Tangent-curve and curve-tangent sections occur frequently in the driving environment because each curve is preceded and succeeded by either a tangent section or another curve section of a highway, and curves along highways are common, especially in locations where there are hills or when highways follow natural rivers. For example, according to Zwahlen (*3*), in Ohio there are more than 18,000 curves along the two-lane rural state highways. Because Ohio has approximately 19,000 mi of highways, of which approximately 1,200 mi are interstate highways, one curve exists for approximately every mile of two-lane rural state highway.

Figure 1 shows the geometric conditions and equations to calculate the peripheral visual detection angles for tangent-curve and curve-tangent sections for a right curve. For the curve-tangent section, two equations had to be developed to calculate the peripheral visual angle. The appropriate equation may be chosen by first calculating the distance *S3*, which is then used to determine the appropriate equation for calculating the peripheral visual detection angle. Because of the optical properties of most retroreflective materials and the

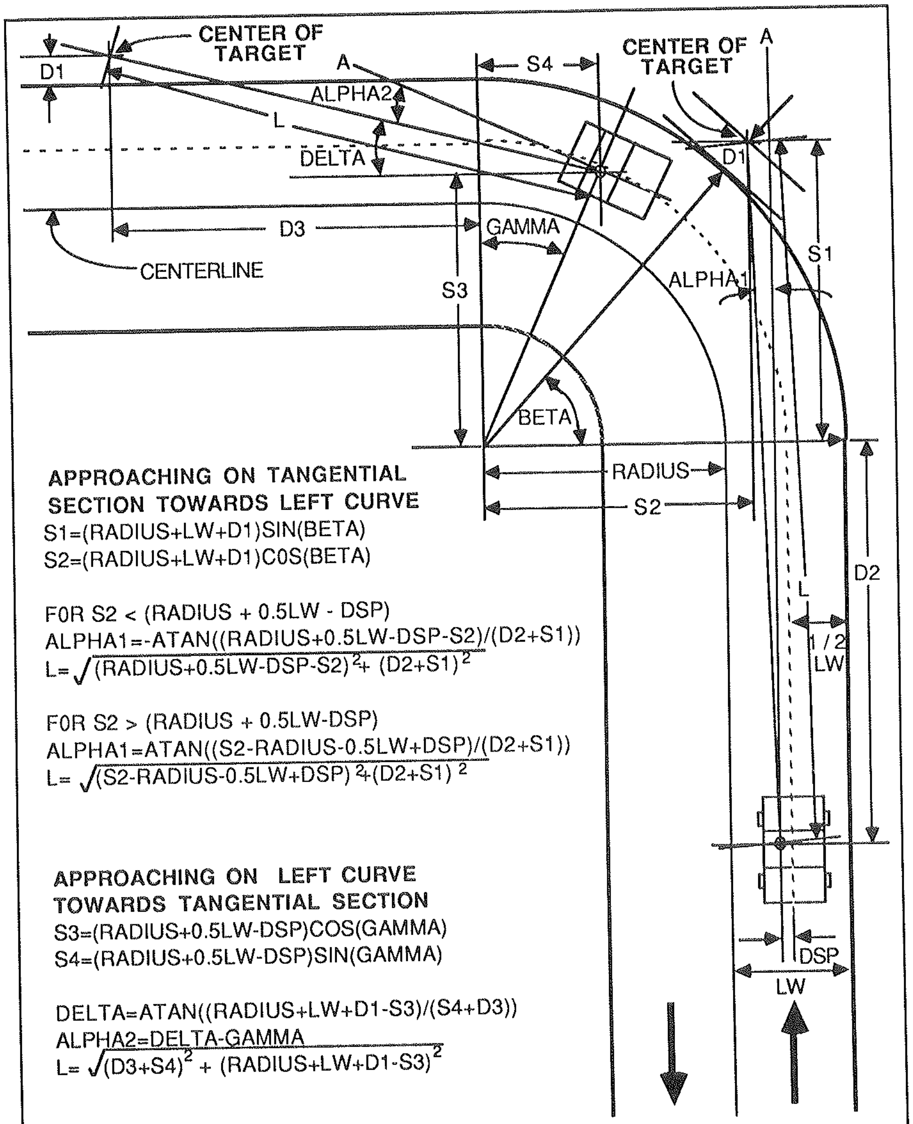


**FIGURE 2 Geometric configuration and equations to calculate the peripheral visual detection angles for tangent-curve and curve-tangent sections for a left curve of a two-lane highway.**

reduced projected areas of the targets, beta and gamma should not exceed 40°.

Figure 2 shows the geometry and equations to calculate the peripheral visual detection angles (*ALPHA1* and *ALPHA2*) and distances ($L$) for the tangent-curve and curve-tangent sections for a left curve. When calculating the peripheral visual detection angle (*ALPHA1*) and distance ($L$) for the tangent-curve section it was necessary to develop two equations from which one equation must be chosen based on the position of the target in the curve. As shown in Figure 2, the distance $S2$ must be calculated before the appropriate set of equations can be chosen because the appropriate formula for the peripheral visual detection angle is based on the magnitude of $S2$. Calculated peripheral visual detection angles to the left of the driver's sagittal plane have negative values and calculated peripheral visual detection angles to the right of the driver's sagittal plane have positive values. Figure 2 also shows an equation to calculate the peripheral visual detection angle (*ALPHA2*) for the curve-tangent section of a left curve. It should be noted that all peripheral visual detection angles for tangent-curve and curve-tangent sections for right curves are measured to the right of the driver's sagittal plane and all angles for tangent-curve and curve-tangent sections for left curves are measured to the left of the driver's sagittal plane, except when $S2 > (RADIUS + 0.5LW - DSP)$ for the tangent-curve section for left curves. In this case, the peripheral visual detection angle is measured to the right of the driver's sagittal plane. A common spreadsheet package (Microsoft Excel) and a graphics package (Cricket Graph) for a Macintosh computer were used to calculate and display the results for selected combinations of the variables present in the model.

According to Zwahlen (*3*), the majority of these curves in the Ohio two-lane rural highway system have a curvature of between 3° and 28° with an average of 12°. Therefore, representative calculations were performed to determine the effect of curvatures of 3°, 12°, and 28° (radii of 1,906, 477, and 204 ft, respectively) on peripheral visual detection angles.

Two lateral offset values on the right-hand side of the driving lane were chosen to represent two typical reflectorized targets that might appear in a driver's peripheral visual field. These targets included a reflectorized license plate of a disabled or abandoned vehicle and a reflectorized roadside warning sign. It was assumed that the disabled vehicle was situated such that the longitudinal center of the vehicle and the reflectorized license plate would be positioned above the right edge line of the highway. It was assumed also that the reflectorized highway warning sign was positioned 12 ft to the right of the edge line (measured from the edge line to the closest edge of the sign) as specified by the *Manual on Uniform Traffic Control Devices* (*4*). Therefore, the center of a 24- × 24-in. roadside warning sign would be 1.4 ft to the right of this mark and the distance measured from the edge of the highway to the center of the sign would be 13.4 ft. To further reduce the number of calculations, it was assumed that: (a) the driver is driving in the center of his or her lane, (b) the driver is driving in a 12-ft wide lane, and (c) the driver's sagittal plane is located 1.25 ft to the left of the vehicle's longitudinal center.

Figures 3 and 4 show the peripheral visual detection angle (alpha) as a function of the distance from the driver's eyes to the reflectorized target (*L*), the curve position angle (beta), the radius of the curve (*RAD*), and the horizontal distance from the right edge of the road to the target of interest (*D1*) for the tangent-curve conditions for right and left curves. The figures show that as a driver approaches the target the peripheral visual detection angles increase for the tangent-curve sections of highway. Figure 4 shows absolute peripheral visual detection angles, because the values for a curve radius of 204 ft (28° curvature) with a curve position angle (beta) of 20° and a distance from the right edge of the road to the target of 13.4 ft is positive and the values for all other conditions are negative.

Figures 5 and 6 show peripheral visual angles as a function of the distance from the driver's eyes to the target of interest, the radius of the curve (*RAD*), the curve completion angle (gamma), and the horizontal distance from the right edge of the highway to the target of interest (*D1*) for the curve-tangent condition for a right and a left curve, respectively. From Figure 5, the peripheral visual detection angles decrease as the distance from the driver's eyes to the target decreases for the investigated conditions. The one exception occurs for a curve radius

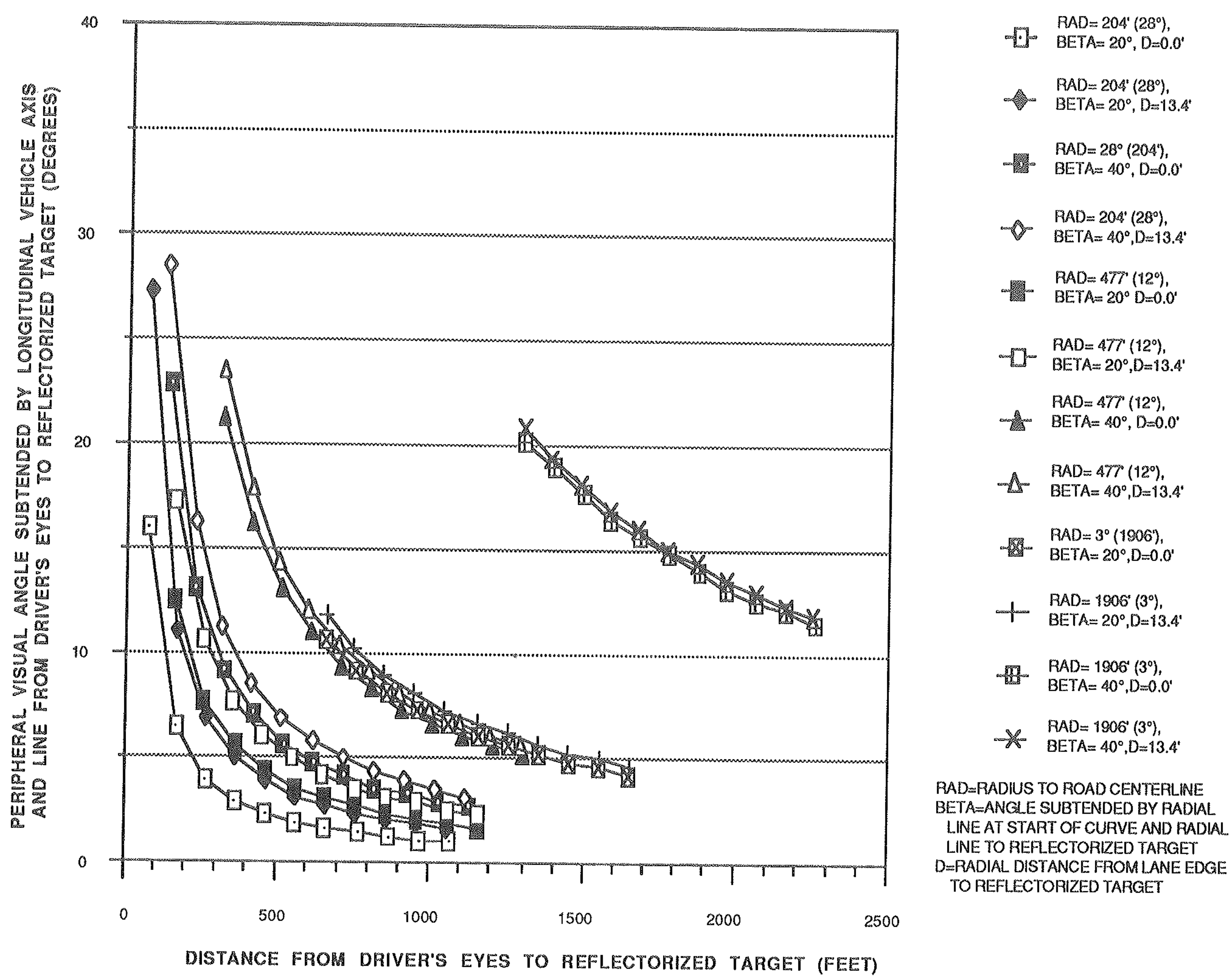


**FIGURE 3 Peripheral detection angles for the tangent-curve section for a right curve of a highway for various distances, curve radii, curve position angles, and lateral distances.**

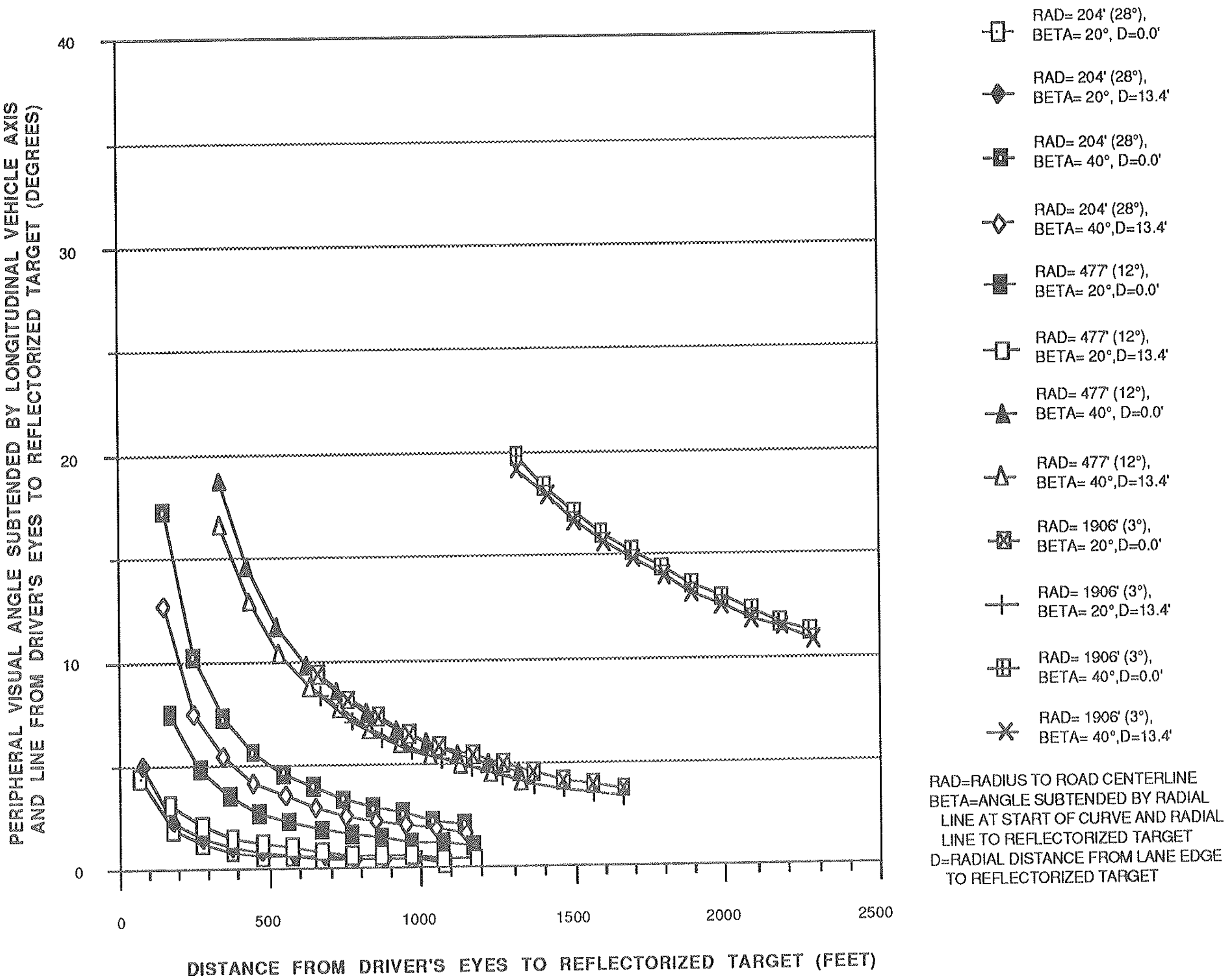


**FIGURE 4 Peripheral detection angles for the tangent-curve section for a left curve of a highway for various distances, curve radii, curve position angles, and lateral distances.**

of 204 ft (28° curvature) with a curve completion angle (gamma) of 20° when the target is located 13.4 ft to the right of the right edge of the highway. For this condition, the peripheral visual detection angle increases as the distance from the driver's eyes to the target decreases.

From the data shown in Figures 3 through 6, it would appear that relatively large peripheral visual detection angles may exist for targets located along or just beyond a curve. However, reviewing the assumptions made in developing the geometric model it was assumed that the direction of a driver's foveal fixation, or line of sight, is along a line parallel to the longitudinal center axis of the car. This assumption may not be valid because a driver fixates upon various targets located ahead of the car in the driving environment. Therefore, it might be necessary to adjust the obtained calculated peripheral visual detection angles according to the experimentally obtained spatial driver eye fixation densities. It should also be noted that only flat and level highways with horizontal curves were considered and vertical curves or combinations of horizontal and vertical curves, which could further increase the magnitude of the peripheral visual detection angles, were not considered.

In a prior study, Zwahlen (*5*) investigated driver eye scanning behavior. Subjects drove on four unlighted 1-mi-long tangent sections of a four-lane interstate highway (Interstate 70 between Ohio State Routes 37 and 79) with a lane width of 12 ft, and on four unlighted right 240-ft radius clover-leaf type entrance/exit ramps (at the intersection of Interstate 70 and Ohio State Route 79) with a lane width of 16 ft, at night and under dry and light rain conditions. Eleven young licensed test drivers who were in good health, had about 20/20 uncorrected vision, and were paid participated in this night driving study. The subjects drove an instrumented car (VW 412, automatic transmission, type 4000 low beams) equipped with an in-car television eye scanning recording system and other electronic equipment. For a more detailed description of the experimental apparatus see the report by Zwahlen (*4*). Each driver drove the ramps and tangent sections in the same order two times.

Figure 7 shows the relative number of eye fixations that occurred in each 1-degree × 1-degree cell within the viewing area for the tangent sections of a four-lane highway. More fixations are focused in the 1-degree × 1-degree cell that is centered 0.5° to the right of the focus of expansion and 0.5°

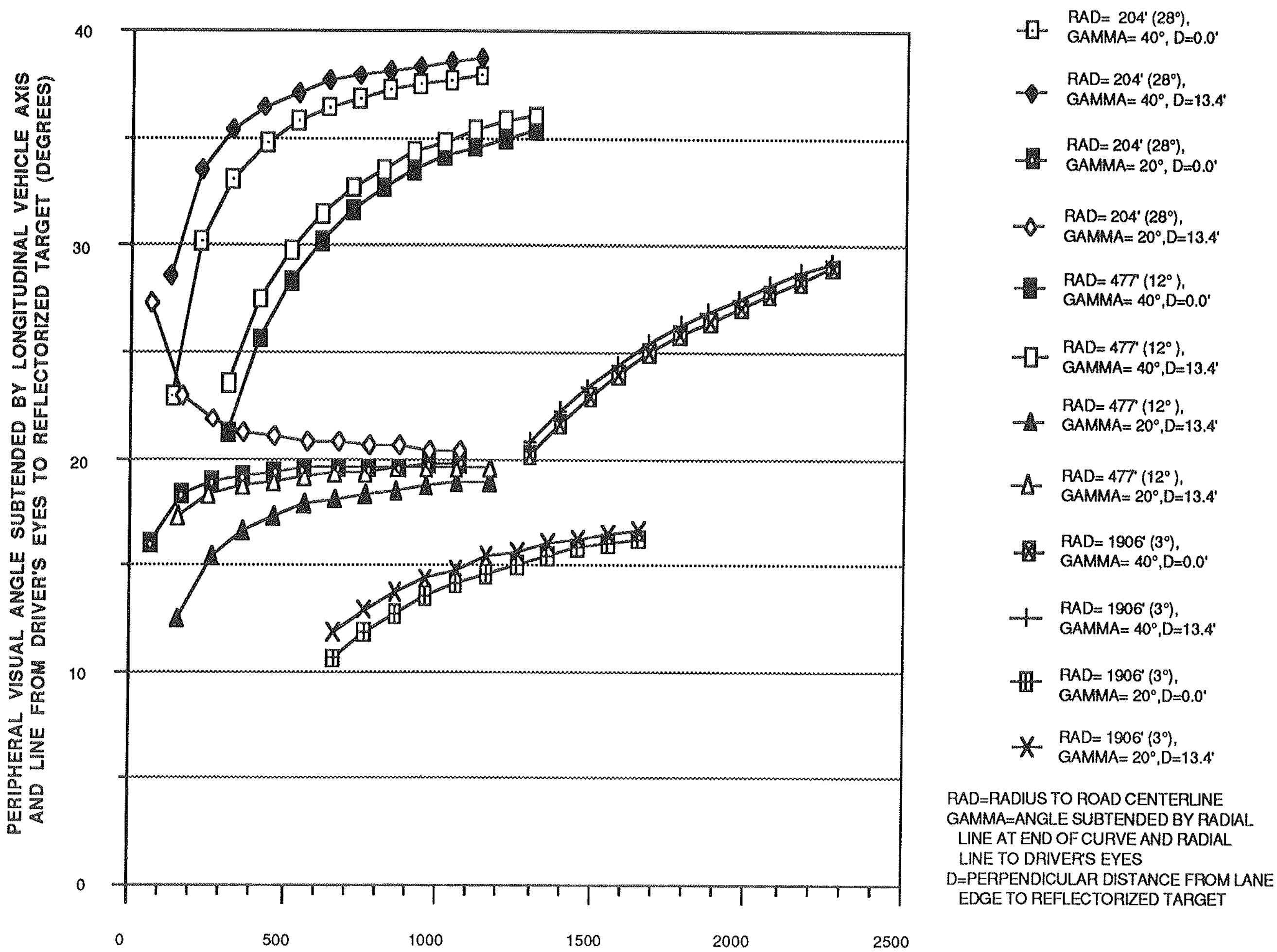


**FIGURE 5 Peripheral detection angles for the curve-tangent section for a right curve of a highway for various distances, curve radii, curve position angles, and lateral distances.**

below the horizon and focus of expansion than in any other cell. In fact, this cell contains 13.5 percent of the eye fixations made by the test drivers. The average of the horizontal eye fixation distribution for the sample size of 11,780 eye fixations is approximately 0.84° to the right of the focus of expansion with a standard deviation of 1.92°.

Figure 8 shows the relative number of eye fixations that occurred in each cell (1° horizontal, 0.95° vertical) within the viewing area for the 240-ft radius right curves. Comparing Figure 8 with Figure 7 for the tangent sections, the eye fixations are dispersed much more in the spatial distribution for the curves. In fact, the cell centered 14.5° to the right of the imaginary focus of expansion and 1.7° below the horizon or imaginary focus of expansion, which contains the most fixations, contains only 3.9 percent of the fixations made by the drivers as they negotiated the 240-ft radius right curves. The average of the horizontal eye fixation distribution for a sample size of 8,884 eye fixations is approximately 12.79° to the right of the imaginary focus of expansion with a standard deviation of 3.79°.

Because a driver's eye scanning behavior consists of a continuous string of discrete eye fixations, there is no way to predict exactly where a driver will look at any instant. There may be a very remote possibility that a driver will, by chance, look directly at an appearing target. However, looking at the spatial distribution of eye fixations it is very unlikely that this will occur, especially for a target that is located 10° or 20° away from the focus of expansion. Figure 7 shows that on tangent sections more than 80 percent of all eye fixations are within a relatively small rectangle extending from −2° left to 3° right of the focus of expansion and from 2° below to 2° above the focus of expansion, or an area of 20° squared. To get some idea about how much the calculated peripheral visual angles should be adjusted to account for the observed horizontal eye fixation distributions, the average and the standard deviation of the horizontal eye fixation distribution were estimated for a 12° and for a 28° left and right curve. The estimates were based on the data presented in Figures 7 and 8, on computer-generated curve driving scene perspectives, and on the assumption that the eye fixation distributions for left curves

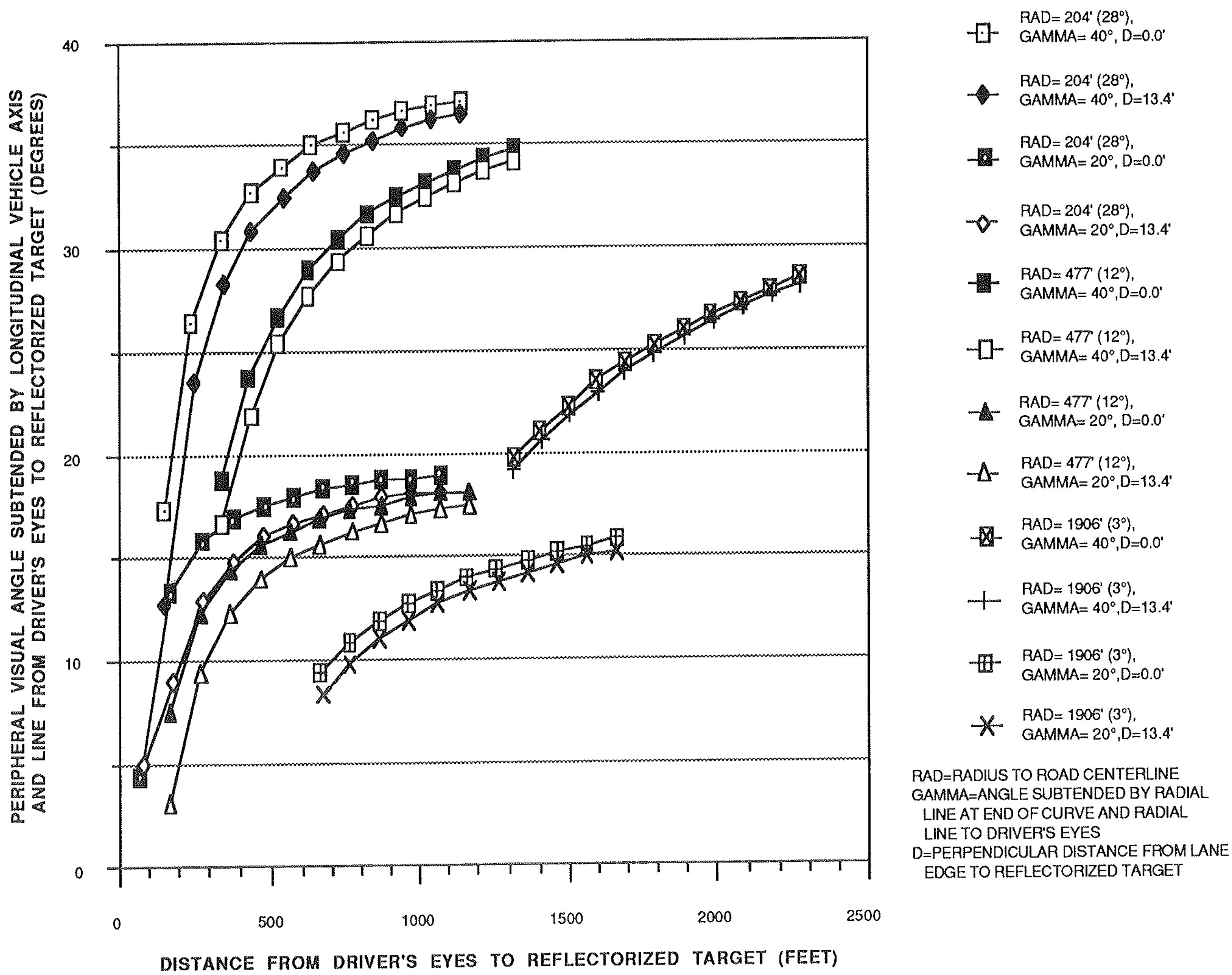


**FIGURE 6 Peripheral detection angles for the curve-tangent section for a left curve of a highway for various distances, curve radii, curve position angles, and lateral distances.**

are approximately symmetric and the same as for right curves. Three horizontal eye fixation location values were then selected (average, average minus one standard deviation, and average plus one standard deviation) to represent horizontal eye fixation locations for the instant when a target becomes first visible in a driver's field of view. Table 1 gives information about the calculated and adjusted peripheral visual detection angles. Table 1 was developed for a selected target distance of 500 ft (the euclidean distance from the driver's eyes to the target of interest). This distance corresponds roughly to the average peripheral visual detection distance minus one standard deviation for a 10° peripheral detection angle and near maximum low beam output (−3° car heading angle) as it was presented by Zwahlen (*1*). Further, Zwahlen (*3*) found average first look or eye fixation distances on curve warning signs on two-lane rural highways at night of about 500 ft. Looking at Figures 3 to 6, one can see that a target distance of less than 500 ft would result in an increase of the calculated peripheral detection angles in some cases and in a decrease of the calculated peripheral detection angles in other cases.

When adjusting the calculated peripheral angles for all tangent-curve sections, an average foveal eye fixation position of 0.84° to the right of the focus of expansion and a standard deviation of 1.92° were used. The curve-tangent sections for 12° right curves were adjusted for an estimated average horizontal foveal eye fixation position of 9.0° to the right of the imaginary focus of expansion and an estimated standard deviation of 3.3°. The curve-tangent sections for 28° right curves were adjusted for an estimated average horizontal foveal eye position of 13.6° to the right of the imaginary focus of expansion and an estimated standard deviation of 4.0°. The curve-tangent sections for 12° left curves were adjusted for an estimated average horizontal foveal eye fixation position of 7.2° to the left of the imaginary focus of expansion and an estimated standard deviation of 3.3°. The curve-tangent sections for 28-degree left curves were adjusted for an estimated average horizontal foveal eye position of 11.2° to the left of the imaginary focus of expansion and an estimated standard deviation of 4.0°. Only curves with 12° and 28° of curvature (radii of 204 and 477 ft, respectively) were selected because no calculated peripheral visual detection angles were available for curves with 3° of curvature (1,906-ft radius) at a distance of 500 ft.

Looking at Table 1, one can see that adjusting the calculated peripheral visual detection angles (based on the estimated average, the estimated average minus one estimated

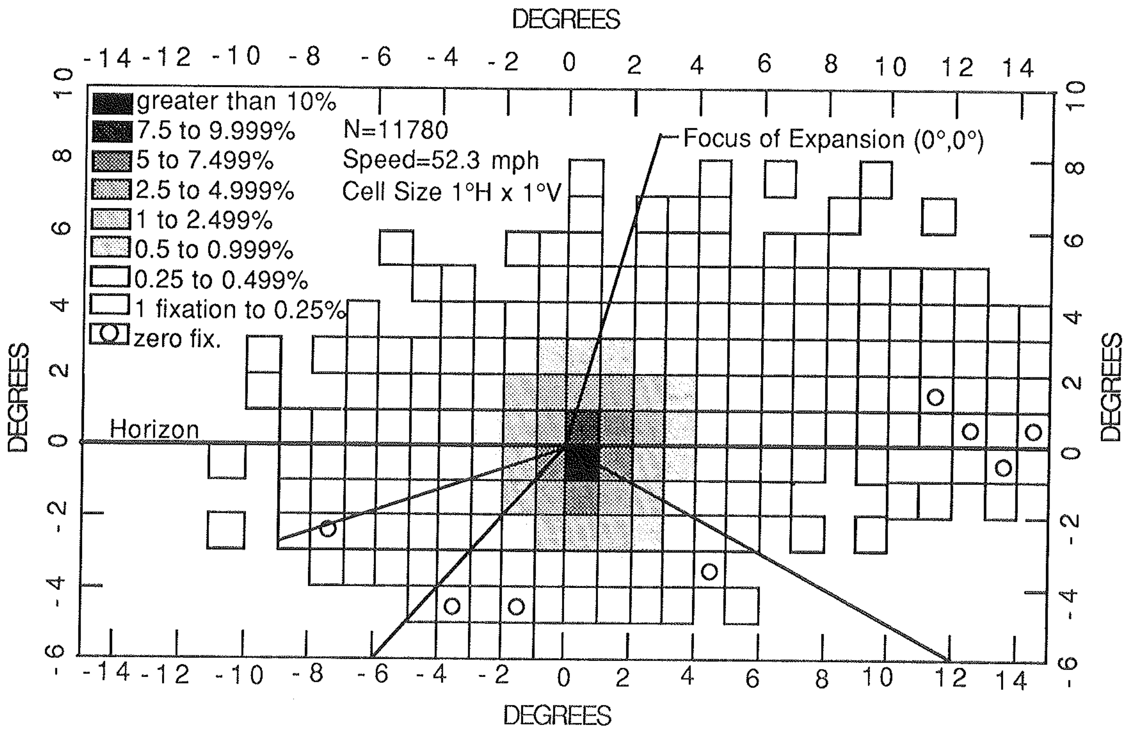


**FIGURE 7** Driver eye fixation pattern for tangent sections of a four-lane interstate highway at night (11 subjects, 91 runs, 13.3 percent of all eye fixations on vehicles ahead).

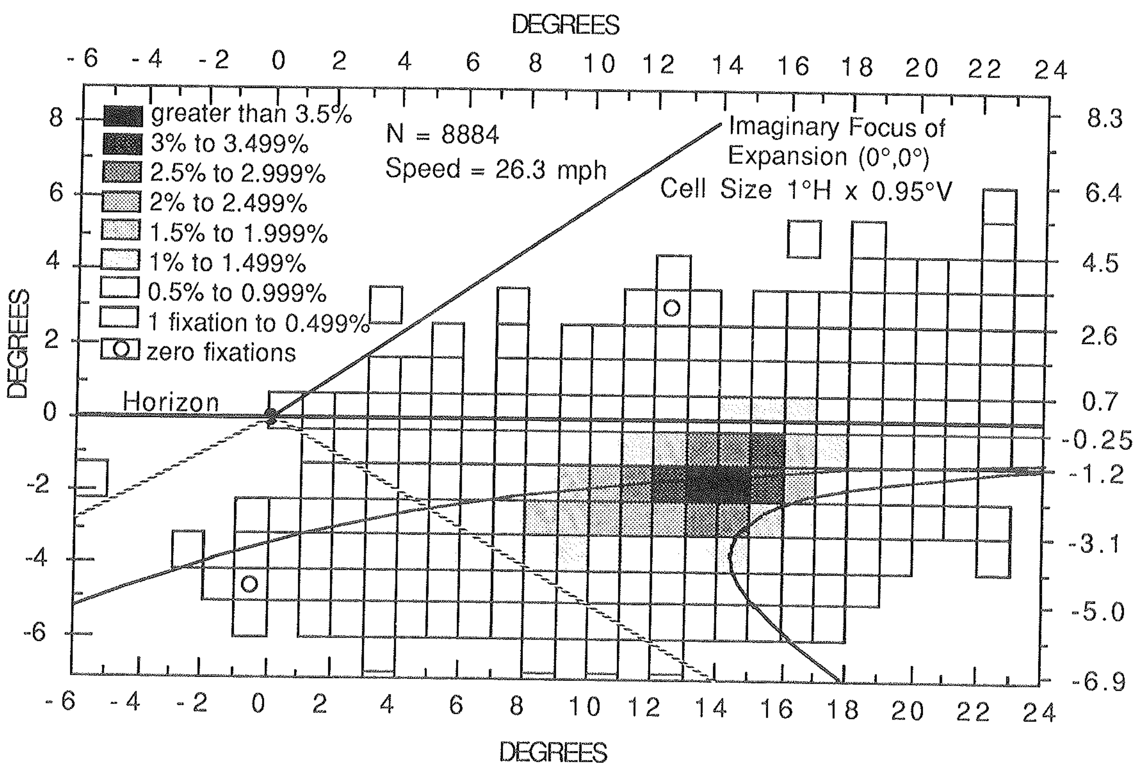


**FIGURE 8** Driver eye fixation pattern for 240-ft radius right-curved entrance/exit ramps at night (11 subjects, 153 runs, 0.7 percent of all eye fixations on vehicles ahead).

TABLE 1 CALCULATED AND ADJUSTED PERIPHERAL VISUAL ANGLES FOR 500-FT VIEWING DISTANCE

| Degree of Curvature | Beta/ Gamma (degrees) | Target Position[a] ($D1$)(ft) | Peripheral Visual Angle: Calculated (degrees) | Adjusted (degrees): $X^b$ | $X - S^c$ | $X + S^d$ |
|---|---|---|---|---|---|---|
| Tangent Right | | | | | | |
| 12 | 40 | 0.0 | 13.5 | 12.7 | 14.6 | 10.8 |
| 12 | 40 | 13.4 | 14.7 | 13.9 | 15.8 | 12.0 |
| 28 | 40 | 0.0 | 6.0 | 5.2 | 7.1 | 3.3 |
| 28 | 40 | 13.4 | 7.2 | 6.4 | 8.3 | 4.5 |
| Curve Right | | | | | | |
| 12 | 20 | 0.0 | 17.5 | 8.5 | 11.8 | 5.2 |
| 12 | 20 | 13.4 | 19.1 | 10.1 | 13.4 | 6.8 |
| 28 | 20 | 0.0 | 19.4 | 5.8 | 9.8 | 1.8 |
| 28 | 20 | 13.4 | 21.0 | 7.4 | 11.4 | 3.4 |
| Tangent Left | | | | | | |
| 12 | 40 | 0.0 | −12.5 | −13.3 | −11.4 | −15.2 |
| 12 | 40 | 13.4 | −11.3 | −12.1 | −10.2 | −14.0 |
| 28 | 40 | 0.0 | −5.0 | −5.8 | −3.9 | −7.7 |
| 28 | 40 | 13.4 | −3.8 | −4.6 | −2.7 | −6.5 |
| Curve Left | | | | | | |
| 12 | 20 | 0.0 | −15.8 | −8.6 | −5.3 | −11.9 |
| 12 | 20 | 13.4 | −14.2 | −7.0 | −3.7 | −10.3 |
| 28 | 20 | 0.0 | −17.7 | −6.5 | −2.5 | −10.5 |
| 28 | 20 | 13.4 | −16.2 | −5.0 | −1.0 | −9.0 |

[a]Target position is from the right edge of the highway.
[b]Adjusted using the estimated average of the horizontal eye fixation distribution.
[c]Adjusted using the estimated average minus one estimated standard deviation of the horizontal eye fixation distribution.
[d]Adjusted using the estimated average plus one estimated standard deviation of the horizontal eye fixation distribution.

standard deviation, and the estimated average plus one estimated standard deviation of the horizontal eye fixation distribution data) resulted in considerable decreases of the magnitude of the angles for about one half of all the cases (curve left and right). For the other half (tangent), the magnitude of the angles was only slightly reduced. However, it should be noted that the magnitude of many of these angles remains relatively large after adjustment.

## PERIPHERAL VISUAL DETECTION CAPABILITIES

An experiment was conducted to assess the conspicuity of a suprathreshold reflective target at night in the field. Some aspects of this study have been reported in Zwahlen (*1*). Two separate groups of subjects were used to study the effects of two different beam output conditions (high candlepower values and moderate candlepower values in the direction of the reflectorized target) and the relative stability and reliability of the experimental results. The first group had 7 subjects (5 males, 2 females) with an average age of 21.1 yrs (standard deviation of 0.9 yrs). This group of subjects had an average of 5.6 yrs driving experience during which they drove an average of 5,000 mi per yr, with standard deviations of 1.9 yrs and 3,000 mi per yr. The second group had 7 subjects (3 males, 4 females) with an average age of 23.5 yrs (standard deviation of 1.7 yrs). The second group had an average of 7.1 yrs driving experience during which they drove an average of 8,700 mi per yr, with standard deviations of 2.2 yrs and 3,300 mi per yr. All of the subjects had normal visual acuity and volunteered their time as subjects. Each subject served as his or her own control.

A 1979 Ford Fairmont was used as the experimental car for the first group of subjects. The headlamps (H4656) were 24 in. above the ground and had a horizontal center-to-center distance of 48 in. The actual established location of the hottest spot for the left low beam was 2° to the right and 2° down. The actual established location of the hottest spot for the right low beam was 1.5° to the right and 1.7° down. The electrical system of the car operated at an average of 13.3 volts. The average distance from the longitudinal vertical center plane of the car to the subject's sagittal plane while in the driver position was 14 in. The average horizontal distance from the headlamps to the subject's eyes was 89 in. and the average subject eye height was 45 in. above the ground.

A 1979 Ford LTD II was the experimental car for the second group of subjects. Its headlamps (GE 4562) were 29 in. above the ground with a vertical center-to-center distance of 46 in. The actual established location of the hottest spot for the left low beam was 2° to the right and 2° down. The actual established location of the hottest spot for the right low beam was 1.5° to the right and 1.7° down. The electrical system of the car operated at an average of 14.1 volts. The distance from the headlamps to the subject's eyes was 97 in. and the average subject eye height was 43 in. above the ground.

A black bicycle was used as the target vehicle. A white license plate was mounted on the front of the bicycle so that the horizontal center of the license plate was 26.8 in. above the paved surface and its reflecting surface made an angle of $-10°$ with the transverse axis of the bicycle to simulate a vehicle parked at a slight angle along the highway. The 6-in. × 12-in. license plate had a reflectivity of 24 CIL (measured 23.5 cd/fc at 0.2° observation angle and $-4°$ entrance angle).

A 75-ft wide, 2,000-ft long section of an abandoned concrete runway located at the edge of the city of Athens, Ohio, was used as the experimental site. A two-lane state highway with moderate traffic was located parallel to and approximately 200 ft away from the runway. A number of luminaires, a few advertising signs, and other light sources were in the subject's field of view, mainly in the left peripheral field. Typical luminance ranges for the 75-ft wide concrete runway surface under low beam illumination were 0.006 to 0.016 fL at 150 ft in the front of the vehicle; 0.004 to 0.009 fL at 300 ft; 0.002 to 0.009 fL at 600 ft; and sky above horizon 0.004 to 0.01 fL, haze, no stars, and no moon (Pritchard photometer, 1° aperture for 150 ft; 20 min aperture for 300 ft, 600 ft, and sky measurements).

There were three approach paths parallel to the runway axis. The front centers of the experimental cars were placed at the zero distance line, vertically above the center line of the runway. Looking forward from the car, Path 1 was 12.5 ft to the left of the runway center line, Path 2 was 6.25 ft to the right of the runway center line, and Path 3 was 25 ft to the right of the runway center line. The inclusion of three paths in the experiment was intended to introduce some uncertainty about the lateral location of the approaching target. The car was then positioned on the runway so that it was heading 3° to the left of the center of the runway ($-3°$ car heading angle) for Group 1 or 10° to the right of the center of the runway (10° car heading angle) for Group 2. The $-3°$ car heading angle produced close to maximum low beam candlepower values in the direction of the reflectorized target and the 10-degree car heading angle produced considerably lower low beam candlepower values in the direction of the reflectorized target. Stakes were placed radially 500 ft away from the car at angles of $-30°$, $-20°$, $-10°$, 0°, 10°, 20°, and 30° with respect to the runway center line to indicate where one movable red dim light (3 ft above the ground) should be positioned as a fixation point to be used by the subjects.

During the experiment, a group of dark-clothed experimenters were positioned at various locations along the side of the runway. Using a flashlight, they signaled the beginning of each trial to an experimenter sitting in the passenger seat of the stationary experimental car. Another experimenter sitting in the car recorded the time of each trial, car voltage, weather conditions, and the subject's responses. The engine of the experimental car was kept idling throughout the experiment. When the experimenter in the car received the signal that the bicycle rider was ready for an approach and the measurement crew was off the runway, the subject was asked to fixate on the dim red light positioned 500 ft ahead of the car at one of the seven selected detection angles. The subject was then instructed to turn on the low beams and be prepared to detect the approaching license plate while continuously fixating his or her eyes on the dim red light. The bicycle rider would approach the stationary car along one of the three approach paths at a constant speed of approximately 10 mph. As soon as the subject had the initial sensation of detection of the target, he or she would switch immediately from low to high beams and keep them on for a few seconds. When the bicycle rider perceived the high beams, he or she would drop a small sandbag on the runway to indicate the detection distance. The measurement crew would then measure the detection distance, pick up the sandbag, and return it to the bicycle rider.

The measurement crew would signal the experimenter in the car indicating the beginning of the next trial after everyone had cleared the runway, the bicycle rider had moved back to the end of the runway, and the bicycle was positioned perpendicular to the runway center line on the correct approach path for the next trial. The correct approach path of the bicycle with the target and fixation point position was checked by the experimenter in the car. Six practice trials were carried out for each subject. This was then followed by the 63 actual trials (7 detection angles × 3 paths × 3 approaches). The experiment required approximately 1 hr and 20 min to complete for each subject.

The independent variables for this experiment were the seven detection angles ($-30°$, $-20°$, $-10°$, 0°, 10°, 20°, and 30° with respect to the runway center line) and the two car heading angles ($-3°$ to the left and 10° to the right). The dependent variable was the detection distance measured in feet. The order of presentation of the peripheral detection angles was according to a latin square design (7 angles, 7 subjects). The 9 observations for each angle (3 paths × 3 replications) were blocked (3 blocks, each path assigned in random order once within each block).

The detection distances obtained for each of the three approach paths were combined because paired $t$-tests indicated that almost all differences among the three paths were not statistically significant at the 0.05 level. The combined results are in Figure 9, which shows averages and standard deviations for the detection distances as functions of the peripheral visual detection angle. From Figure 9, one can see that the average detection distances decrease considerably as the peripheral visual detection angle increases. At a peripheral visual detection angle of 10° the average detection distance was between 54.2 and 59.2 percent of the average foveal detection distance for the $-3$-degree car heading angle and between 47.3 and 55.6 percent of the average foveal detection distance for the 10° car heading angle. At a peripheral visual detection angle of 20° the average peripheral detection distances were between 35.7 and 36.0 percent of the average foveal detection distance for the $-3°$ car heading angle and between 35.6 and 37.9 percent of the average foveal detection distance for the 10° car heading angle. At a peripheral angle of 30° the average peripheral detection distances were between 25.1 and 27.0 percent of the average foveal detection distance for the $-3°$ car heading angle and between 28.2 and 32.6 percent of the average foveal detection distance for the 10° car heading angle. As expected, the average detection distances obtained for the 10° car heading angle, or the much

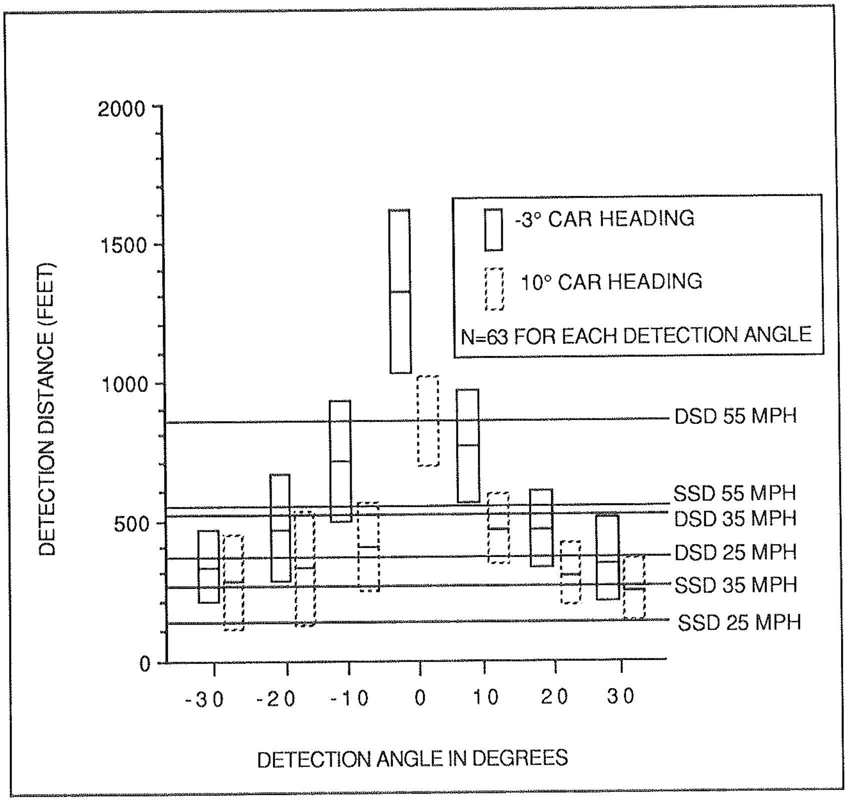


**FIGURE 9 Averages and standard deviations of the measured peripheral visual detection distances for a −3-degree and 10-degree car heading angle compared with selected stopping sight distances and lower values of selected decision sight distance ranges.**

lower candlepower values of the low beams, are considerably shorter than the average detection distances obtained for the −3° car heading angle.

The average detection distances obtained in this study and shown in Figure 9 can be further evaluated from a safety point of view. Comparing the peripheral detection distances with a recommended stopping sight distance of 563 ft for a speed of 55 mph, only the average detection distances for the foveal and the 10° peripheral detection angles exceed the recommended stopping sight distance for the −3° car heading angle. If the recommended stopping sight distance of 563 ft for a speed of 55 mph is compared to the average detection distance for the 10° car heading angle, then the recommended stopping sight distance is larger than all of the average detection distances except the average detection distance for the 0° (foveal) detection angle. If the idealized average detection distances obtained in this study are reduced by 50 percent to adjust for driver alertness and expectancy, older drivers, information acquisition and information processing load while driving, somewhat degraded environmental visual conditions, dirty windshield, and dirty headlamps, then only the 50 percent reduced average detection distance for the 0° peripheral angle (foveal detection) for the −3° car heading angle exceeds the stopping sight distance for a speed of 55 mph. Comparing the average detection distances acquired for the −3° car heading angle after they are reduced by 50 percent with a stopping sight distance of 263 ft for a speed of 35 mph, the reduced average detection distances are larger than the stopping sight distance for only the foveal detection and the 10-degree peripheral visual detection angles. Further, comparing the average detection distances acquired for the 10° car heading angle, only the reduced average detection distances for the 0° peripheral visual detection angle are larger than the stopping sight distance. In fact, once the average detection distances are reduced by 50 percent they are so small that for the 10° car heading angle the reduced average detection distances for the 30° peripheral angles are approximately equal to or slightly smaller than the stopping sight distance of 137 ft for a speed of 25 mph.

McGee et al. (*6*) recommended decision sight distances, i.e., the distances a driver needs to perceive a potentially hazardous situation and react efficiently to the impending danger, of 375 to 525 ft for a speed of 25 mph, 525 to 725 ft for a speed of 35 mph, and 875 to 1,150 ft for a speed of 55 mph. Comparing the smaller of each of these distances with the detection distances obtained in this study, the average detection distances are greater than the minimum decision sight distance for a design speed of 55 mph for only the foveal detection angle with the −3° car heading angle (near optimal low beam candlepower conditions). As the peripheral visual detection angle is increased, the average detection distances decline so rapidly that for the relatively small peripheral detection angle of 10° the average detection distances for 10° car heading angle are less than the decision sight distance for a speed of 35 mph. The average detection distances for a peripheral detection angle of 30° are as much as 130 ft less than the decision sight distance for a speed of 25 mph. If the decision sight distances are compared to the detection distances reduced by 50 percent, then the decision sight distance for a speed of 55 mph is larger than all of the 50 percent reduced average detection distances for both the $-3°$ and 10° car heading angles. Comparing the decision sight distances for a speed of 25 mph to the average detection distances reduced by 50 percent, only the detection distance for the 0° visual detection angle for the 10° car heading angle is larger than the decision sight distance for a speed of 25 mph. Similarly, only the average detection distances obtained for the 0° and 10° peripheral visual detection angles for the $-3°$ car heading angle are equal to or larger than the minimum decision sight distance for a speed of 25 mph. The much shorter 10° car heading detection distance results might be more applicable to the peripheral visual detection of a reflectorized target in the highway environment than the $-3°$ car heading detection distance results. The longitudinal direction of the car and its beams is such that the candlepower values of the beams in the direction of the reflectorized target are probably reduced considerably in a situation where a reflectorized target first appears in a driver's peripheral visual field.

Zwahlen (*7*) has shown that the multiples of threshold that a driver needs to detect a reflectorized target, such as a bicycle pedal, increase very rapidly as the peripheral visual detection angle is increased. The multiples of threshold are proportional to the specific intensity of a reflectorized target. Therefore, if, for a given peripheral visual detection angle, an average detection distance equal to the average foveal detection distance is desired, the specific intensity or reflectivity of the retroreflective target would have to increase appropriately, assuming the environmental and beam conditions remain the same.

Paired *t*-tests were performed to determine whether or not the average peripheral detection distances for the left side (peripheral visual detection angles of $-30°$, $-20°$, and $-10°$) could be assumed to be equal to the corresponding average peripheral detection distances for the right side (peripheral visual detection angles of 10°, 20°, and 30°). For both the $-3°$ and the 10° car heading angles, the average peripheral detection distances for the 10-degree peripheral visual detection angle for the left side were about 9.2 to 17.6 percent shorter (statistically significant at the 0.05 level) than the average peripheral detection distances for the right side. This can be partially explained by noting that there was a highway with moderate traffic located about 200 ft away on the left parallel to the airport runway. Therefore, there were more light sources (luminaires, advertising signs, etc.) in the left peripheral field of view (less uniform dark background). The average peripheral detection distances for the 20° and 30° peripheral visual detection angles were of about equal magnitude and were not statistically different.

## CONCLUSIONS

Using Ohio as an example, it has been shown that curve-tangent and tangent-curve sections occur fairly frequently along two-lane rural highways, especially in hilly regions. Therefore, relatively large peripheral visual detection angles (up to 15° or more from a driver's foveal eye fixation point or line of sight) could be quite common for reflectorized targets that become visible for the first time in the periphery of a driver's visual field. The field study to assess the peripheral visual detection capability of drivers at night, or the conspicuity of suprathreshold reflectorized targets, produced visual detection distances with nearly ideal subjects under fairly ideal and well-controlled conditions. The results show that the peripheral visual detection ability, or the detection distances for suprathreshold reflectorized targets, decrease considerably as the peripheral visual detection angle increases. The decrease of the visual detection distances in the periphery can, however, be offset by increasing the reflectivity or specific intensity of the retroreflective target. It is, therefore, recommended that in cases where a reflectorized target will become visible for the first time most likely in the periphery of a driver's visual field and where there is a need for early detection, the reflectivity of the target should be increased to assure timely recognition, information processing, and decision making, and appropriate control actions.

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# Redesign and Evaluation of Selected Work Zone Sign Symbols

JEFFREY F. PANIATI

**A study of traffic sign symbols was completed recently at the FHWA Turner-Fairbank Highway Research Center in McLean, Va. As part of this research, four work zone traffic control warning signs (Pavement Width Transition, Flagger Ahead, Low Shoulder, and Uneven Pavement) were selected for redesign. Several alternatives for each sign were tested in a laboratory experiment. Legibility distance, comprehension, and preference data were collected for each alternative. The results of the study indicate that the current symbolic designs for Pavement Width Transition and Flagger Ahead are superior to the alternatives tested and should be retained. The current symbolic sign for low shoulders was found to be inadequate (22 percent comprehension), but an acceptable alternative was not found. Given the infrequent use of this sign and the difficulty of communicating this concept to the driver, the use of the alphabetic Low Shoulder sign is recommended. For the Uneven Pavement sign, an improved symbolic design was identified and is recommended.**

In recent years symbolic designs have been used to improve the communication ability of traffic signs. Symbols have the advantage of being able to convey messages more concisely than traditional alphabetic signs. Most symbols have become well known and readily understood. However, certain symbols, because of their infrequent use or abstract meaning, do not serve their intended purpose. The FHWA receives and responds routinely to complaints from motorists about signs that are unsatisfactory.

A study of the legibility and comprehension of symbolic traffic signs was conducted at the FHWA Turner-Fairbank Highway Research Center (TFHRC) in McLean, Virginia. This paper discusses the results of the symbolic redesign portion of the research. In this part of the study, alternatives were tested for four work zone traffic control warning symbols identified as needing redesign. The four are the symbols for Pavement Width Transition, Flagger Ahead, Uneven Pavement, and Low Shoulder.

## BACKGROUND

A brief review of the literature for each of the signs selected for redesign is presented below. The discussion includes an outline of the problems with the existing design, the findings from other research studies, and the alternatives that were included in this research study.

Federal Highway Administration, Office of Safety and Traffic Operations R&D, 6300 Georgetown Pike, McLean, Va. 22101.

### Pavement Width Transition

The Pavement Width Transition sign is a symbolic sign currently included in the *Manual on Uniform Traffic Control Devices* (MUTCD) (*1*). Although this sign does not have a word alternative, the MUTCD does require that the Right [Left] Lane Ends alphabetic warning sign be used in advance of the Pavement Width Transition symbolic sign. Complaints received from motorists indicate that some drivers misinterpret the thick black lines on the sign as representing the lanes and not the edge of the road. On other signs, thick bold lines are used to represent the lanes.

Pietrucha and Knoblauch (*2*) studied this sign in their research on sign comprehension. The initial comprehension tests indicated that the sign shown in Figure 1—Alternative 1 outperformed the existing sign. However, subsequent simulator and controlled vehicle tests provided conflicting results. The authors recommended that additional research be conducted to resolve this conflict.

Other studies have also reported conflicting data on the effectiveness of the existing sign. A study by Hulbert et al. (*3*) on the understandability of traffic control devices found that 87 percent of the subjects questioned could correctly identify this sign, but a study by Koppa and Guseman (*4*) found only 65 percent comprehension.

Two of the more promising alternatives developed by Pietrucha and Knoblauch along with the existing design were selected for this experiment. These designs are shown in Figure 1.

### Flagger Ahead

Currently, the MUTCD contains both alphabetic and symbolic versions of the Flagger Ahead sign. While both signs are fairly well understood by motorists, there has been some thought to modify the symbol to show the flagger holding a paddle instead of a flag. This is because Revision 4 of the MUTCD specifies the use of the STOP/SLOW sign paddle as the primary hand signaling device. The proposed version of the paddle sign is shown in Figure 2—Alternative 3.

An FHWA in-house research study (Alicandri, Walker, and Roberts, unpublished data) used the FHWA driving simulator to evaluate the existing sign and the proposed paddle sign. In this study, 95 percent of the drivers who saw the flag sign remembered it, but only 45 percent of the drivers who saw the paddle sign remembered it. In questioning the subjects on the meaning of both signs, the authors found that


**FIGURE 1** Alternative signs for Pavement Width Transition.

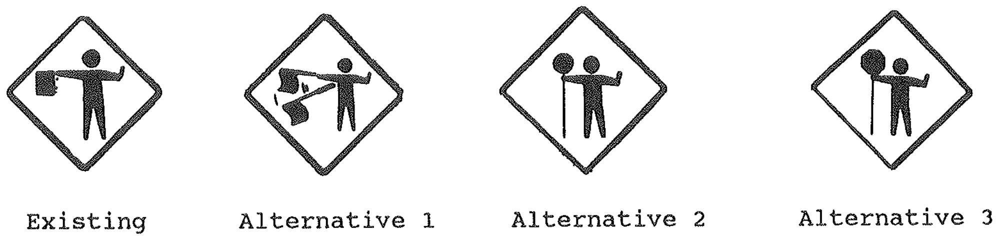



**FIGURE 2** Alternatives for Flagger Ahead sign.

the flag sign had 100 percent correct answers and the paddle sign had 42 percent correct. From the data, it appeared that many subjects identified the paddle sign as the sign held by school crossing guards and expected to see it near a school zone. The octagonal shape of the paddle led many subjects to respond that they expected a stop ahead.

Pietrucha and Knoblauch (*2*) also examined the design of the Flagger Ahead sign. They studied several alternatives for the sign, but the paddle sign was not among them. They found that the best alternative was one that showed apparent motion, Figure 2—Alternative 1, and recommended that further study of this design be undertaken for inclusion in the MUTCD.

The existing sign along with three alternatives was studied in this experiment. The first alternative was the paddle sign developed by Alicandri et al. The results of their research showed that drivers often interpreted the sign to mean Stop because of the Stop-sign-shaped paddle. As a way to address this problem, a second paddle sign (Figure 2—Alternative 2) was developed using a round paddle. Finally, the alternative recommended by Pietrucha and Knoblauch was included. The existing sign and the alternatives are shown in Figure 2.

**Low Shoulder**

The MUTCD contains only an alphabetic version of the Low Shoulder sign. However, *Standard Highway Signs* (*5*) contains a symbolic version of the sign that is used widely. It is shown in Figure 3.

Pietrucha and Knoblauch (*2*) found a total lack of motorist understanding of this sign, as evidenced by only 4 percent correct responses. They tested a number of alternative designs with the best having 52 percent comprehension.

The existing Low Shoulder symbolic sign was also included in a research study by Wilson and Williams (*6*). They found that only 34 percent of the subjects could identify the sign correctly. The authors concluded that there was too much confusion with the Uneven Pavement symbolic sign and recommended that other alternatives be tested.

The development of an acceptable symbol for Low Shoulder is very difficult, as shown by results from previous studies. This study included a modification of Pietrucha and Knoblauch's best alternative (Figure 3—Alternative 2), which adds texture to the shoulder to denote a different surface. It is hypothesized that this will assist in differentiating the sign

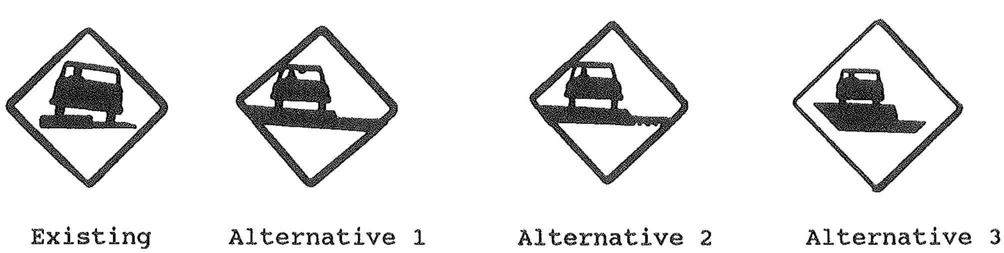



**FIGURE 3** Alternatives for Low Shoulder sign.

Alternative 1 Alternative 2 Alternative 3

**FIGURE 4 Alternatives for Uneven Pavement sign.**

from the Uneven Pavement symbolic sign and provide the driver with an added cue. A nontextured version of the sign (Figure 3—Alternative 1) was also included. A third alternative (Figure 3—Alternative 3), the design used currently in Texas for shoulder drop-off (*7*), was also included in the study.

### Uneven Pavement

The MUTCD contains only an alphabetic version of the Uneven Pavement sign. While there is no symbolic version of the sign in the MUTCD, some states have developed one (Figure 4—Alternatives 1 and 3).

The existing versions of the symbolic sign are often confused with Low Shoulder symbolic signs. Several research studies have examined alternatives for this sign. Pietrucha and Knoblauch (*2*) studied the comprehension of several alternatives for the Uneven Pavement symbolic sign. They found that two signs, Figure 4—Alternative 3 and Figure 4—Alternative 2, provided good levels of comprehension (76 percent and 73 percent, respectively). They concluded that further testing was warranted.

Wilson and Williams (*6*) evaluated the design shown in Figure 4—Alternative 1 and found that 14 percent of the subjects responded correctly, with an additional 63 percent close to the correct response. They also found a large number of different responses for this sign, which demonstrates serious misunderstanding, and considerable confusion with the Low Shoulder symbolic sign. They suggested that alternative symbolic designs be tested.

For this research study, the best alternative from the Pietrucha and Knoblauch study was included with the two other designs currently in use. These test alternatives are shown in Figure 4.

## EXPERIMENTAL METHODOLOGY

### Measures of Effectiveness

Many different studies involving a variety of techniques have evaluated symbol signs. Each of these techniques has its advantages and disadvantages and each provides different information about the sign. However, many of the studies use only one measure of effectiveness and examine only one aspect of the sign. In their review of methods for the evaluation of traffic signs, Dewar and Ells (*8*) state, "For the complete evaluation of traffic signs a single method will not be adequate. . . ." In a later paper on the same topic, Dewar and Ells (*9*) conclude that the development and evaluation of any symbol should include an assessment of comprehension, legibility distance, and preference.

These three measures of effectiveness were used in this study. The legibility data indicated which symbol can be discriminated most effectively by the driver, sign comprehension data indicated how easily the different symbols can be understood, and preference data measured driver satisfaction with the various alternatives.

Dewar (*10*) surveyed sign design experts and practicing traffic engineers to determine the relative importance they placed on the various criteria used to evaluate traffic sign symbols. Legibility distance and understandability or comprehension were two of the criteria included. Dewar found that understandability was judged as being more important than legibility distance. The findings from Dewar's research should be kept in mind when interpreting the results from this study.

Preference was not included in Dewar's study, but generally it is viewed as of low importance. It is most useful as a "tie-breaker" when the legibility distance and comprehension measures fail to discriminate between alternatives. Preference rating is also a way of generating feedback from the subjects.

### Test Apparatus

A test apparatus was developed to collect legibility distance and comprehension data for a large number of symbol signs. The apparatus consisted of a zoom lens mounted on a MAST random access slide projector. The lens had a 10.5:1 ratio allowing slides to be projected on a rear projection screen as they would appear at distances ranging from 110 to 1,000 ft. At any given distance, the visual image on the retina from a projected sign corresponded to the image that would be formed by the same sign in an actual driving situation.

A Textronics development system controlled the system and allowed slides to be selected randomly. Computer controlled servos controlled the zoom ratio and zoom lens aperture. This allowed the apparent size of the image to be enlarged while maintaining proper sign brightness.

The experiment was conducted in the Human Factors Laboratory at the FHWA TFHRC. The test apparatus is shown in Figure 5.

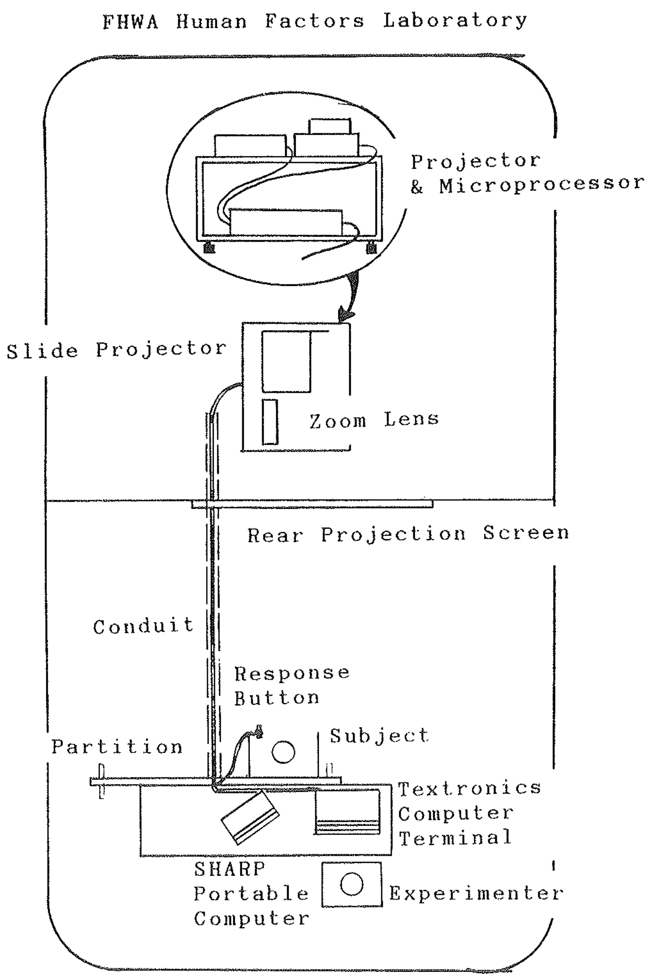



**FIGURE 5 Test apparatus for laboratory experiment.**

### Test Stimuli

The stimuli consisted of slides of the sign images made using the FHWA graphics system. This system produces high quality graphics (resolution of 1,024 pixels × 780 pixels). An RGB camera recorded the graphic image, and the slides generated were reduced photographically to provide small images with high quality resolution.

### Test Protocol

#### *Briefing/Vision Screening*

Before participating in this study, subjects were briefed on the requirements of the experiment and were given a far visual acuity test using the orthorater test instrument. A Snellen value of 20/40 was determined as the cutoff level for subject participation. This is the level that most states require for driver licensing.

#### *Dynamic Legibility Test*

A dynamic recognition distance test was developed to assess the legibility distance of the signs. This test consisted of presenting the subject with a randomly selected slide beginning at a simulated distance of 1,000 ft and moving forward at a rate equivalent to a driving speed of 44 ft/sec (30 mi/hr). The subject was given a handheld button and instructed to depress it when the symbol on the projected sign could be identified. Since the legibility distance of the sign was of interest, the subject was told to press the button when the features of the symbol could be described, even if the meaning was not clear.

When the button was pressed the slide was extinguished immediately and the subject was asked to describe the slide. If the response was correct, the distance was noted by the experimenter and the next trial was initiated. If the answer was incorrect, the trial resumed from the point of interruption.

An acceptable response was one that indicated that the essential features on the sign could be distinguished. For example, a typical acceptable answer for an Uneven Pavement sign might be "a car, on a road, tipped, with the lower side to the left." The response "a car on a road" would not be acceptable.

#### *Comprehension Test*

The symbolic signs were presented randomly to a subject, one at a time, to assess comprehension. The signs were displayed at a large size and for as long as the subject required to respond. Subject responses were classified into one of the following categories:

- *Correct:* The response demonstrated a clear understanding of the intended meaning of the sign;
- *Substantially Correct:* The response was not exact, but did indicate a reasonable understanding of the sign meaning; and
- *Incorrect:* The response demonstrated a lack of understanding of the intended meaning.

#### *Preference/Debriefing*

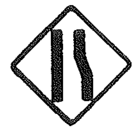
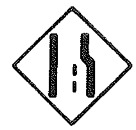
Subject preference data were collected at the conclusion of the study. Each subject was given the meaning of the four proposed symbol signs (Uneven Pavement, Low Shoulder, Flagger Ahead, and Right Lane Ends) and asked to rank the alternatives from best to worst.

### Subject Group

Many of the changes that occur in driver behavior have been found to correlate negatively with age. This negative correlation is the result of deterioration in the physical capabilities of the eye, decreases in cognitive performance, and delay in the motor response functions. To study the effects of age, the subject population was stratified into two age brackets—under 45 and over 55. Each group contained 16 subjects of which half were male and half were female.

The test sample included 32 drivers from 20 years old to 68 years old with an average age of 47. The average age for the "old" group was 61 and for the "young" group was 33. The subjects were paid $30 for their participation in this study.

TABLE 1 PAVEMENT WIDTH TRANSITION DATA

| |  |  |  |
|---|---|---|---|
| Measure of Effectiveness | Exist | Alt-1 | Alt-2 |
| Mean Legibility Distance (ft.) | 519 | 513 | 518 |
| Comprehension - Correct | 22% | 38% | 22% |
| - Sub/Correct | 66% | 59% | 72% |
| - Incorrect | 12% | 6% | 6% |
| Preference Rating | 60 | 59 | 73 |

## RESULTS AND ANALYSIS

### Pavement Width Transition

The existing design and two alternative symbolic designs for the Pavement Width Transition sign were tested. The results are presented by measure of effectiveness in Table 1 and discussed below.

#### *Legibility Distance*

An analysis of variance (ANOVA) with repeated measures was performed on the legibility data. This analysis showed a statistically significant difference between the age groups ($p = 0.02$), but did not indicate any differences between the sign alternatives ($p = 0.94$). A post-hoc test for multiple comparisons was performed to assess the difference between individual sign pairs. The Ryan-Elnot-Gabriel-Welsch (REGW) test was selected because it is considered one of the more powerful tests of this type. The results of the REGW test showed no statistically significant difference between any of the alternatives.

The short legibility distances for the three alternatives are not surprising if one considers their "long-distance" appearance. From far away, the signs appear the same. Subjects respond to the thick bold exterior lines when asked to describe the sign.

#### *Comprehension*

The comprehension data for the Pavement Width Transition sign were analyzed using the Cochran $Q$-test. This test is a nonparametric statistical test that determines whether three or more matched sets of frequencies differ significantly among themselves. To use this test the comprehension responses were reclassified as either correct or incorrect. The correct category included both correct and substantially correct responses. Performing the required computations resulted in $Q = 1.14$ and $p > 0.70$. This result indicates that the frequency of each response was the same except for chance differences.

The reason for testing different alternatives for the existing sign was to address the problem of drivers mistaking the heavy bold lines for the lanes instead of the edge of the road. From the responses given in this experiment, it appears that this confusion does exist with some drivers. The four incorrect responses given to the existing sign were all of this type. The alternatives tested helped somewhat, but they did not eliminate the problem. Both alternatives had two incorrect responses of the same type.

#### *Preference*

The preference rating was determined by summing the ranks for each alternative. The symbol with the lowest preference rating was the preferred alternative. Another nonparametric statistical test, the Friedman two-way analysis of variance by ranks test, was used to evaluate the preference data. The Friedman test resulted in a $x^2$ statistic of 3.81 giving a $p > 0.10$. This indicates that the preference ranking produced no statistically significant difference among alternatives.

### Flagger Ahead

For the Flagger Ahead sign, three alternative symbolic designs were tested along with the existing design. The results for each alternative are presented by measure of effectiveness in Table 2 and discussed below.

#### *Legibility Distance*

The ANOVA with repeated measures indicated a statistically significant difference between the age groups ($p = 0.003$) and among the sign alternatives ($p = 0.000$). The REGW post-hoc test for multiple comparisons was used to determine which sign pairs were statistically different. The results showed that

TABLE 2 FLAGGER AHEAD DATA

| |  |  |  |  |
|---|---|---|---|---|
| Measure of Effectiveness | Exist | Alt-1 | Alt-2 | Alt-3 |
| Mean Legibility Distance (ft.) | 532 | 367 | 460 | 448 |
| Comprehension - Correct | 84% | 63% | 31% | 47% |
| - Sub/Correct | 6% | 22% | 13% | 22% |
| - Incorrect | 9% | 16% | 56% | 31% |
| Preference Rating | 61 | 79 | 84 | 96 |

at a level of confidence of 0.05, all the sign pairs were different except for Alternatives 3 and 4.

The legibility distance results indicated a clear difference among the three basic types of Flagger Ahead signs studied—the standard flagger sign, the flagger waving the flag, and the flagger holding a paddle. The legibility distance data did not show a statistically significant difference between the two types of paddle signs, round (Alternative 2) and octagonal (Alternative 3). The "long-distance" appearance of these signs is similar.

The reasons for the difference among the three basic types were evident during the testing. The standard sign was a familiar shape that was easily described at a far distance by the majority of the subjects. Alternatives 2 and 3 had shorter legibility distances because of the additional distance required by the subject to resolve the paddle. Alternative 1 had the shortest legibility distance because the sign had to be fairly large for the subject to discriminate the two flags used to depict motion.

### *Comprehension*

The comprehension data for the Flagger Ahead sign were analyzed using the Cochran $Q$-test and resulted in $Q = 27.72$ and $p < 0.001$. This indicates that there is a statistically significant difference among alternatives.

Examining the comprehension data for the four designs, the two paddle alternatives had a high percentage of incorrect responses (56 percent and 31 percent). This was because, in large part, subjects believed these signs represented a school crossing guard. The two flag alternatives gave much better comprehension results. The results for these two designs were comparable, with the existing sign getting slightly fewer incorrect responses and more completely correct responses.

### *Preference*

Evaluating the preference data using the Friedman two-way analysis of variance by ranks test resulted in a $x^2$ statistic of 11.89 giving a $p < 0.01$. This result indicates a statistically significant difference among the alternatives. From the sum of the ranks, the existing sign was preferred over any of the other alternatives. Many subjects were confused with the two paddle alternatives and indicated that the motion alternative was more distracting than helpful. They believed the existing sign clearly indicated the presence of a flagger ahead.

When asked if they would prefer the use of a word sign instead of their highest ranked symbol, the subjects overwhelmingly (91 percent) preferred the use of a symbol. Conversations with subjects also indicated that the primary problem they had with this sign was not one of misunderstanding, but of misuse. The Flagger Ahead sign is too often displayed when there is no flagger or even any construction to be found.

## Low Shoulder

Three alternative symbolic designs were tested along with the existing design. The results for each alternative are presented by measure of effectiveness in Table 3 and discussed below.

### *Legibility Distance*

The ANOVA with repeated measures indicated a statistically significant difference among the age groups ($p = 0.004$). However, there was no statistically significant difference between the sign alternatives ($p = 0.824$). The REGW post-hoc test for multiple comparisons also showed that none of the alternatives was significantly different from another based on legibility distance.

The short legibility distances for this sign result from two factors: (a) the amount of detail that must be resolved, and (b) the complexity of the message. All of the alternatives required the subject to distinguish that one part of the road was lower than the other. Resolving detail of a small scale requires a fairly large image size. In addition, the concept of low shoulder is both unfamiliar and complex. This contributed to its short legibility distance.

TABLE 3 LOW SHOULDER DATA

| |  |  |  |  |
|---|---|---|---|---|
| Measure of Effectiveness | Exist | Alt-1 | Alt-2 | Alt-3 |
| Mean Legibility Distance (ft.) | 217 | 202 | 206 | 210 |
| Comprehension - Correct | 16% | 25% | 34% | 34% |
| - Sub/Correct | 6% | 16% | 25% | 6% |
| - Incorrect | 78% | 59% | 41% | 59% |
| Preference Rating | 93 | 87 | 62 | 78 |

### *Comprehension*

The comprehension data for the Low Shoulder sign were analyzed using the Cochran $Q$-test and resulted in $Q = 13.50$ and $p < 0.01$. This indicates a statistically significant difference among alternatives.

The data clearly show that the Low Shoulder sign with the textured shoulder (Alternative 2) provided the best comprehension, while the existing design was the least understood alternative. The low comprehension for the current design is because drivers mistake it for the Uneven Pavement sign. Of the 25 incorrect responses to this sign, 19 (76 percent) were because of this confusion. Alternatives 1 and 3, while better than the existing design, still showed a significant amount of confusion between Uneven Pavement and Low Shoulder (10 and 6, respectively).

Alternative 2 had only one response of ". . . uneven pavement." The texturing of the shoulder is the main reason for the low amount of confusion. However, Alternative 2 still had more than 40 percent incorrect responses. Many of the incorrect responses to this sign involved a reference to the shoulder's being soft. While the subjects recognized the shoulder and understood that there was a hazard associated with it, they did not comprehend correctly the type of hazard.

### *Preference*

Analyzing the preference data with the Friedman two-way analysis of variance by ranks test resulted in a $x^2$ statistic of 10.24 and a $p < 0.02$. This result indicates a statistically significant difference between the alternatives. The sum of the ranks demonstrated a preference for Alternative 2, the design with the textured shoulder. Many people indicated that the texturing gave them a cue that it was a shoulder and not a traveled lane.

The subjects also showed a strong dislike for the existing sign because of the Uneven Pavement/Low Shoulder confusion. Alternative 1 was ranked low because many people believed the drop-off to be too severe.

The problem of communicating the low shoulder concept was emphasized by the low percentage of drivers (34 percent) who said they preferred their highest-rated symbolic alternative to a word sign. Many subjects believed that the Low Shoulder symbolic sign would be seen too infrequently and they would be more comfortable with a word message.

## **Uneven Pavement**

Three alternative symbolic designs for the Uneven Pavement sign were tested. There is no existing symbolic design in the MUTCD. The results for each alternative are presented by measure of effectiveness in Table 4 and discussed below.

### *Legibility Distance*

The ANOVA with repeated measures showed no statistically significant difference between the age groups ($p = 0.07$). However, there was a statistically significant difference among the sign alternatives ($p = 0.94$). The REGW post-hoc test for multiple comparisons indicated that this difference was between Alternative 3 and the other two alternatives. There was no statistically significant difference between Alternatives 1 and 2.

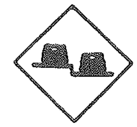
As with the Low Shoulder sign, the legibility distances for the Uneven Pavement alternatives are quite short. The problems associated with the two signs are similar.

The legibility distances for the Uneven Pavement Alternatives 1 and 2 were nearly identical, which is not surprising given their similar designs. The use of two cars in Alternative 3 seemed to contribute to its shorter legibility distance. The two cars did not accentuate the difference in pavement elevation; they only added more complexity to an already difficult sign.

### *Comprehension*

The Cochran $Q$-statistic for the Uneven Pavement sign was $Q = 4.50$ resulting in a $p > 0.10$. This indicates that there

TABLE 4 UNEVEN PAVEMENT DATA

| |  |  |  |
|---|---|---|---|
| Measure of Effectiveness | Alt-1 | Alt-2 | Alt-3 |
| Mean Legibility Distance (ft.) | 216 | 222 | 151 |
| Comprehension - Correct | 50% | 66% | 41% |
| - Sub/Correct | 19% | 22% | 28% |
| - Incorrect | 31% | 13% | 31% |
| Preference Rating | 56 | 58 | 78 |

was no difference in the frequency of responses among the three alternatives. However, the data for the three alternatives show that Alternative 2 had only four incorrect responses and Alternatives 1 and 3 had 10 incorrect responses. None of the alternatives had much confusion with the Low Shoulder sign.

*Preference*

The results of the Friedman two-way analysis of variance by ranks test indicated a statistically significant difference among alternatives. The $x^2$ value of 9.25 results in a $p < 0.01$. The rankings indicated a dislike for Alternative 3. Many of the subjects believed that this sign could represent an uneven road or it could just as easily be misinterpreted to mean ". . . use two lanes" or ". . . two-lane roadway ahead."

It appears that drivers prefer a symbolic sign as opposed to a word sign. Sixty-six percent of the subjects believed that their highest ranked symbolic sign was preferable to an alphabetic sign.

## CONCLUSIONS AND RECOMMENDATIONS

### Pavement Width Transition

No statistically significant difference was found among the three alternatives tested. While the number of incorrect responses was reduced with both of the new designs, the reduction was not statistically significant. The existing design had good comprehension, with 88 percent correct or essentially correct responses. Retention of the existing design is recommended.

### Flagger Ahead

The existing flagger design performed significantly better than any of the alternative designs for all three measures of effectiveness. It was described correctly at longer legibility distances, had the highest comprehension (90 percent correct), and was clearly preferred by the subjects.

The results of this study indicate that there is nothing to be gained and much to be lost by switching from the current design to one showing a paddle instead of the flag. An effective symbol design must get the intended message to the driver clearly and concisely; the current design does just that.

### Low Shoulder

The results of the legibility distance and comprehension tests illustrate the difficulties in symbolically representing the low shoulder concept. None of the alternatives tested performed adequately. The existing design had the lowest comprehension (22 percent correct) and the most confusions with the Uneven Pavement sign. Alternative 2 had the best comprehension (59 percent correct) and had few confusions with the Uneven Pavement sign, but still did not perform at an acceptable level. The preference data highlighted this fact. While Alternative 2 was the preferred design, the majority (66 percent) of the subjects said they preferred a word sign to any of the alternatives.

Given the infrequent use of this sign and the difficulty in communicating the Low Shoulder concept to the driver in a symbolic form, the current symbolic design should be withdrawn and only the word alternative used. Although the word sign was not included in this study, one can assume that the comprehension of this sign (at least by English-speaking, literate drivers) would be significantly higher than the 22 percent comprehension of the current symbolic design. As noted earlier, Dewar (*10*) found comprehension to be the most important evaluation criterion for traffic sign symbols.

### Uneven Pavement

The results of this study indicate that Alternative 2 is the best choice. This alternative had the highest comprehension (88 percent correct) and the longest legibility distance. Contrary to the results for the Low Shoulder sign, the majority (66 percent) of the subjects preferred a symbol for this sign.

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# Supplemental Interchange Signing and Driver Control Behavior

JOSEPH E. HUMMER

**A study was conducted to determine the effect on driver behavior of supplemental interchange signing on rural freeways, including various combinations of tourist-oriented attraction and service signs. Thirty-six test subjects "drove" the FHWA driving simulator over a 40-mi freeway course with 14 interchanges. Subjects were requested to scan the signs presented on an interchange approach and to exit the freeway if they saw a sign for the attraction or service they were seeking. Data were collected on the speed, acceleration pattern, and lateral placement pattern of the simulated vehicles on interchange approaches and on the distances from the signs at which the subjects recognized the logo or legend they were seeking. Supplemental signing in addition to that already permitted on a rural freeway interchange approach was generally detrimental to driver control behavior. Field tests should be conducted to confirm the behavior effects. The design of the supplemental attraction signs used in the simulation may have contributed to the changes in driver behavior, so designs that include different color schemes or that have picture logos as well as legends may be worth testing. Driver age, driver sex, and the number of supplemental service signs were other variables controlled in the experiment that were associated with control behavior differences and should be included in any future testing.**

Inadequate in-trip directional information on attractions available to motorists along rural freeways has received attention recently from transportation professionals. The problem can be summarized as follows (*1*):

> With the exception of major traffic generators, which qualify as destinations on standard or supplemental guide signs, information needs concerning attractions have normally been satisfied by billboards and hard copy information sources. However, the cost of adequate billboards for small, non-profit or local government owned attractions may be prohibitive, especially in areas with limited legal sign space. Hard copy sources suffer from uncertain and unreliable distribution mechanisms and are often less than optimum in fulfilling needs for direction rather than selection. The satisfaction of this group of information needs thus represents one of the major deficiencies of the current information systems.

A possible solution to the problem has been proposed by several states. These states want to install up to two new signboards on the freeway right-of-way before selected rural freeway interchanges and remove current supplemental guide signing. The proposed new signing concept is shown in Figure 1. Each new signboard, referred to hereafter as "attraction sign," would contain up to four attraction names or logos and a message informing the motorist of the proper exit to use. One attraction sign would contain names or logos of traffic generators such as business districts, medical facilities (other than hospitals), or government installations and would be blue with white lettering. The second attraction sign would contain the names or logos of cultural, recreational, and historical facilities and would be brown with white lettering. Each attraction name or logo would fit into an area 3 ft high and 5 ft wide. The sign, including border but not support, would be 10.5 ft high and 13 ft wide. The proposed attraction signs would be erected in advance of the specific service logo signs permitted under federal regulations (*2*) for camping, lodging, food, and fuel with a minimum spacing between supplemental signs of 800 ft. A maximum of 18 business names or logos are now permitted on the four service logo signs. One possible arrangement of the new attraction and other signs near a typical rural interchange is shown in Figure 2. Smaller versions of these attraction signs, with directional arrows and mileage to the attraction, would be erected near freeway ramp terminals for attractions not visible from the ramp.

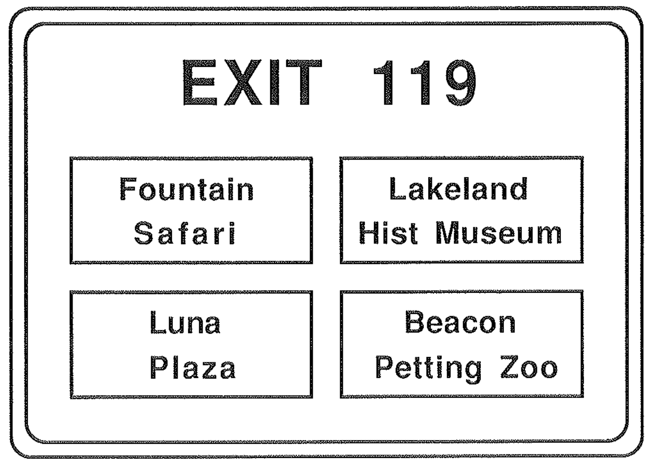


**FIGURE 1 Typical proposed "attraction" sign.**

The driver safety effects of supplemental attraction signs at rural freeway interchanges were investigated in a study for the FHWA (*3*). The objectives of the research were to assess the need for and to narrow the range of variables to be considered in a future field study of attraction signs. A large number of sign-related variables that may affect driver safety were examined. This paper summarizes the results of that examination.

School of Civil Engineering, Purdue University, West Lafayette, Ind. 47907. Current affiliation: Department of Civil Engineering, University of North Carolina at Charlotte, Charlotte, N.C. 28223.

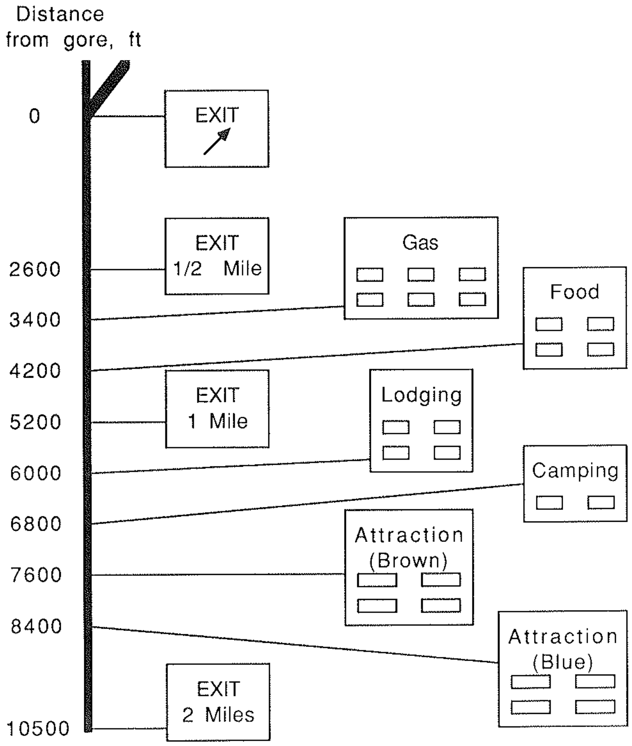


**FIGURE 2 Example sign sequence and spacing in advance of rural freeway interchange including supplemental service and proposed attraction signs.**

## BACKGROUND

The literature was examined for clues on the impact of the propsed attraction signs on driver behavior. From the current regulation on such signs, human factors theory, installations of analogous types of signs, and past installations of attraction signs, it appears that there may be an effect on driver safety from the proposed signs, but the extent of the effect remains unknown.

*The Manual on Uniform Traffic Control Devices* (MUTCD) contains specific regulations on supplemental guide signs (*4*). The MUTCD first specifies the exact appearance of a supplemental guide sign. Then, the MUTCD warns of a potential driver information overload problem from a profusion of such signs and places limits on their use. The proposed attraction signs violate both the specified appearance and usage guidelines; therefore, the concern expressed regarding an impact on driver performance from such signs may be justified.

The human factors literature does not offer a solution to the problem of determining the safety impact of the proposed signs. Driver information overload is a concern regarding signs placed near freeway interchanges, but information overload is difficult to define and it is not yet possible to calculate the magnitude of information overload from theory.

Past studies have been conducted on the effects on driver safety of freeway guide signs, supplemental service logo signs, and advertising signs. Longer messages on freeway guide signs have been shown to degrade driver performance in laboratory and field studies (*5–9*). However, the optimum message length varies widely with sign function and placement. Research on the number of signs before a freeway exit has been inconclusive. Studies of the installation of service logo signs in Ohio (*10*) and Virginia (*11*) and gasoline logo shields in Vermont (*12*) have found no major impacts on rural freeway operations. Controversy remains over the effects of the number of roadside advertising signs on driver behavior, with most recent research indicating that a greater density of advertising signs leads to more driver operation problems.

Vermont and Nebraska, among other states, have installed supplemental signs for attractions on major roads. Vermont's program involves a system of smaller signs along non-freeway roads (*13*). A typical sign has several attraction panels with a standard category symbol, the attraction name, the distance to the attraction, and a directional arrow. The sign is part of a sequence of such signs placed 200 ft apart before an intersection. Driver information overload was a concern of Vermont officials, and field tests made with signs with five attraction panels showed that those signs were unsatisfactory. A limit of three panels per sign was adopted and the standard category symbol was made optional.

Nebraska's program involves installation of signs on rural freeways (*14*). A large sign containing the logo and a legend for a particular attraction is placed near the interchange to be used, while a similar sign is placed 50 mi from the interchange. No attempt was made to measure the driver safety impact of the signs during the extensive evaluation of the program.

## EXPERIMENTAL DESIGN

Since the potential driver safety impact from the proposed attraction signs cannot be identified with the available theory or by analogy to similar sign installations, an experiment was conducted on the FHWA driving simulator (HYSIM). HYSIM models real-world driving conditions in the laboratory with a stationary automobile in which a test subject manipulates vehicle controls in response to a continuously updated roadway view projected on a screen. HYSIM was an advantageous way to conduct the experiment because it offers high degrees of test subject safety and experimental control. Limitations to the accuracy with which HYSIM models real-world driving were mitigated and the results showed that the limitations did not adversely affect the study objectives.

The measures of effectiveness (MOEs) selected for the experiment were measures of driver performance rather than mental load. MOEs were selected because they were related directly to the study objectives, reliable, feasible, and of value in reaching conclusions. Five MOEs were selected:

- Speed differential (the difference between a driver's speed near an experimental sign of interest and the mean speed for all drivers),
- Number of erratic maneuvers (steering far off course or missing an exit),
- Acceleration noise (the standard deviation of acceleration measured at points near an experimental sign),
- Lateral placement (the standard deviation of the center of the vehicle relative to the center of the lane measured at points near an experimental sign), and
- Recognition distance (distance between an experimental

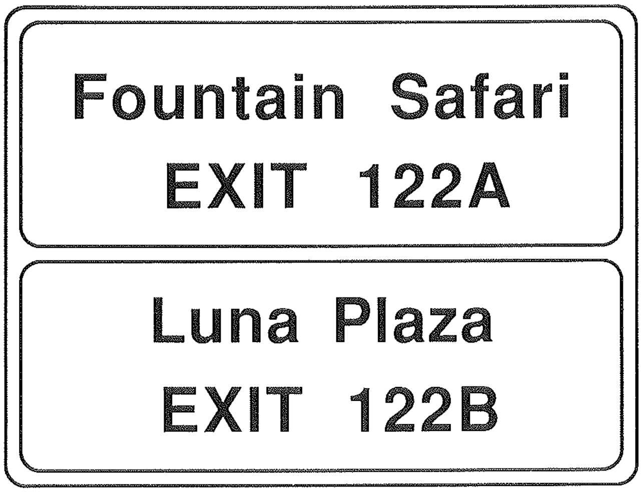


**FIGURE 3 Typical current MUTCD supplemental sign.**

sign and the point at which drivers signaled their recognition of the message on the sign).

Variables under systematic control of the experimenter included the age and sex of the driver, the presence of a pair of the proposed attraction signs versus a current MUTCD supplemental sign (shown in Figure 3), the number of service logo signs, and the number of logos per attraction sign (i.e., two per sign versus four per sign). Testing was conducted during the summer of 1985. The basic instructions given to the 36 paid test subjects were to scan the guide signing presented before a freeway interchange for a specified destination "cue" and to exit where the signs showed the destination was located. The HYSIM driving course consisted of 40 mi of simulated rural freeway with 14 interchanges, 9 of which a test subject used as the experiment progressed. Standard diamond and cloverleaf interchange designs were used and the results for each design type were analyzed separately. The signs were projected from slides produced on an advanced computerized color graphics system and were similar in appearance to actual highway signs. Fictitious two-syllable attraction names rather than pictures or logos were used on the attraction signs to eliminate bias from particularly famous or vague logos.

Eighteen different signs were used for data collection. However, each combination of MOE, cue type, and interchange type was analyzed as a separate experiment. The design of these "separate experiments" was factorial, that is, several different independent variables were controlled and the effects investigated at each of two or more categories. The basic hypothesis tested statistically through the entire effort was that there was no difference in an MOE between different categories of a particular factor (or several interacting factors). Rejection of this hypothesis for a particular combination of variables, MOE, cue type, and interchange type meant that the sign or driver characteristic(s) may influence performance over the ranges tested.

Analysis of variance (ANOVA) was used to test the hypotheses. ANOVA was chosen because it examines interactions among the variables as well as the separate main effect of each variable. For example, for the speed MOE, the "presence of new attraction signs" and "number of service signs" may have proven insignificant by themselves, but a significant interaction between them may have been the basis for the conclusion that there was a driver performance effect when both variables were at extreme points.

## RESULTS

Results from a questionnaire given to the subjects after their simulated drive showed that the subjects thought the experiment was credible. Most test subjects thought the simulation was realistic, the sign legibility was good, the effort required to "drive" 40 mi was about right, and the instructions were easy to understand.

ANOVA was applied to 56 combinations of cue and interchange type, variable main effect, and MOE. A 0.05 level of significance was used, meaning that the hypothesis was rejected only when the chance of a mistaken rejection was equal to or less than 5 percent. The data presented in Table 1 show that the hypothesis was rejected in 16 cases. In many of these cases the hypothesis would have been rejected at very low levels of significance.

Comparisons of the average values of MOE between levels of variables for which the hypothesis was rejected are shown in Figures 4 through 13. Several trends emerged from the comparisons. First, the hypothesis was rejected several times for the driver age variable because of poorer performances by younger and older drivers, for the driver sex variable because of more negative speed differentials for female drivers, for the attraction sign variable because of poorer performances in response to the proposed attraction signs, and for the number of service signs variable because of poorer performance when four signs were presented. Second, rejection of a hypothesis indicated worsened but not necessarily unsafe driver behavior. For example, the basic hypothesis was rejected for the driver age variable and the speed differential MOE when the subject was given an attraction cue and exited at a diamond interchange. The average MOE values for this case showed (Figure 4), however, that the average speed differential for the 50 years and older group was only about 2 mph below that for the other age groups. Third, an anomalous trend was apparent in the case of the attraction sign type variable and the lateral placement MOE with an attraction cue at a diamond interchange. It was expected that the hypothesis was rejected due to greater lateral placement values in response to the proposed attraction sign. However, the plot for the case shown in Figure 9 indicated that the hypothesis was rejected because of greater lateral placement values in response to the current supplemental signs. Analysis of the lateral placement data revealed that the cause of the anomaly was one particular interchange through which almost all test subjects drove with considerably lower lateral placement values than any other. There are several possible explanations for the unexpected lateral placement results at the interchange but no particular explanation was proven.

Interactions between variables were also examined during the statistical analysis. All combinations of two and three variables were analyzed using ANOVA. The hypothesis was rejected for only eight of the several hundred interactions examined. One of those, the type of attraction sign and the

TABLE 1 ANOVA RESULTS FOR MAIN EFFECTS

| Experiment Part | Independent Variable | MOE: Speed Differential | MOE: Lateral Placement | MOE: Acceleration Noise | MOE: Recognition Distance |
|---|---|---|---|---|---|
| 1 (Attraction cue, diamond interchange) | Age | .0001[1] | *[2] | .0035 | .0081 |
| | Sex | .0345 | * | * | * |
| | Attractions Signs | .0049 | .0024 | .0108 | .0001 |
| | Service Signs | * | .0085 | * | * |
| | Logos/messages per sign | * | * | * | * |
| 2 (Attraction cue cloverleaf interchange) | Age | * | * | * | * |
| | Sex | .0384 | * | * | * |
| | Attraction Signs | .0132 | * | * | .0001 |
| 3 (City and attraction cues) | Age | .0017 | * | * | NA[3] |
| | Sex | * | * | * | NA |
| | Attraction Signs | * | * | * | NA |
| | Reinforcement | * | * | * | NA |
| 4 (Service cue, diamond interchange) | Age | .0001 | * | .0385 | NA |
| | Sex | * | * | * | NA |
| | Attraction Signs | * | * | * | NA |
| | Service Signs | * | * | .0365 | NA |

1. A decimal value means that the basic hypothesis was not rejected at a .05 level of significance but would be rejected at the indicated level of significance (e.g., age affected speed differential significantly in experiment part 1).

2. The symbol "*" means that the basic hypothesis was rejected at a .05 level of significance (e.g., age did not significantly affect lateral placement in experiment part 1).

3. The symbol "NA" means that the results were not analyzed or that the analysis was not applicable.

number of service signs for the lateral placement MOE, was related to the main effect anomaly discussed above. Six of the remaining seven interactions with rejected hypotheses were related closely to the significant main effects discussed above. For example, the hypothesis was rejected for the driver sex and type of attraction sign interaction for the speed differential MOE because the basic hypothesis was also rejected for both of the component variable main effects. The analysis of interactions between variables revealed little new information from which to draw conclusions.

The erratic maneuver MOE was not statistically analyzed because of the small sample size. Only 13 erratic maneuvers were recorded out of 324 observations near interchanges. A qualitative analysis of the erratic maneuvers failed to uncover a trend.

A number of subjects preferred the blue over the brown attraction signs. Ten of the 36 subjects indicated such a preference on the post-test questionnaire independent of any particular question and unsolicited by the experimenter. The data do not show a correlation between this preference and performance, however. The difference in average recognition distance between blue and brown attraction signs of interest was small. A *t*-test of these data showed no significant difference and the hypothesis was not rejected.

Data were analyzed which showed a difference in driver performance when drivers viewed signs including "logo" pictures as compared to signs with word messages only. Subjects reported substantially more difficulty recognizing word message attraction signs than logo service signs on the post-test questionnaire. Blue service signs were recognized, on average, at a greater distance than attraction signs. A *t*-test showed a significant difference and the hypothesis was rejected. This result must be tempered considerably by the fact that the service signs used for comparison were gasoline signs with logos of national distinction. Nonetheless, the target recognition value of the picture logo was clear.

## CONCLUSIONS

Conclusions were drawn with the original scope of the study in mind. The study was undertaken to assess the need for and to narrow the range of variables to be considered in a future study.

If serious consideration is given to using attraction signs, a field study to evaluate the proposed attraction signs should be undertaken. The types of supplemental attraction signs viewed made a difference in driver performance for two types

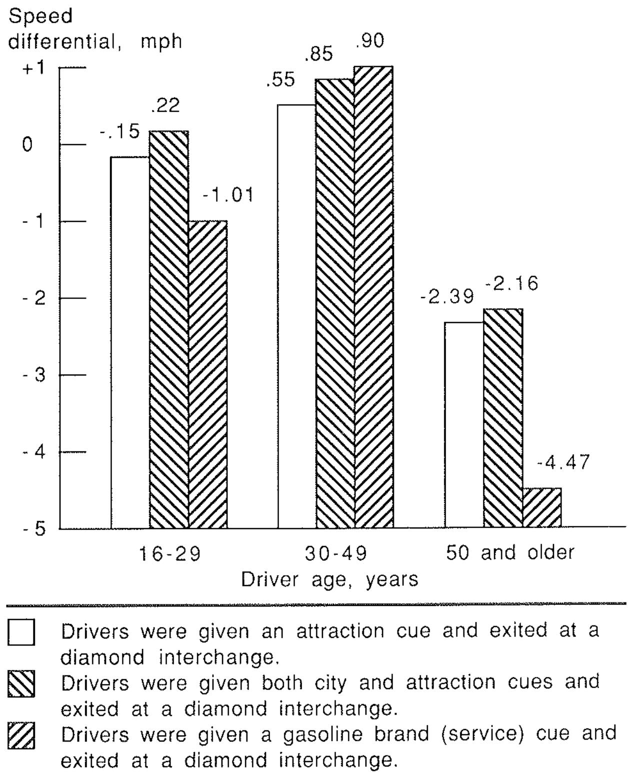


**FIGURE 4 Average values of speed differential for cases with statistically significant driver age effects.**

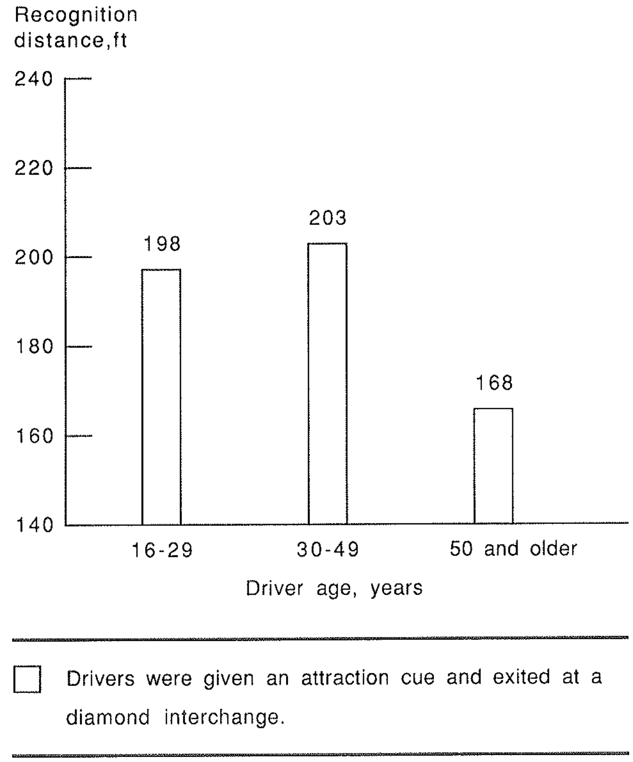


**FIGURE 6 Average values of recognition distance for cases with statistically significant driver age effects.**

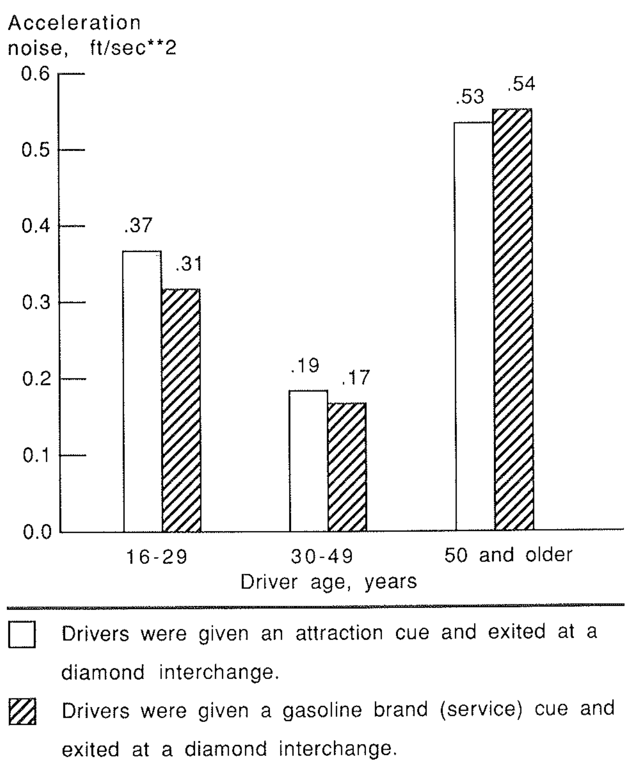


**FIGURE 5 Average values of acceleration noise for cases with statistically significant driver age effects.**

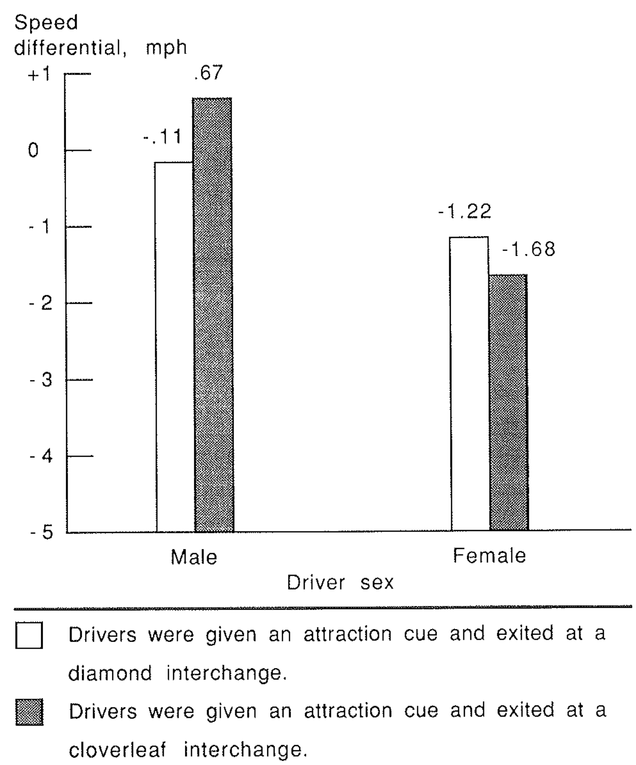


**FIGURE 7 Average values of speed differential for cases with statistically significant driver sex effects.**

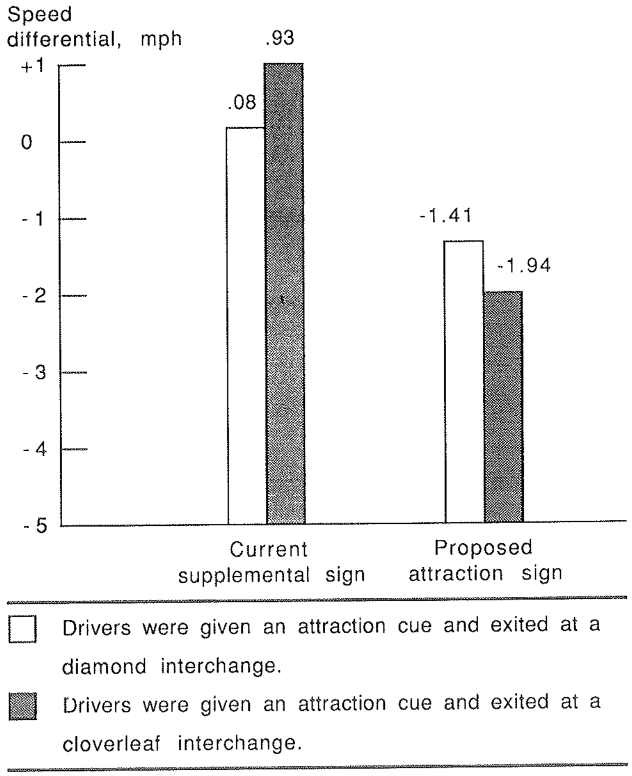


**FIGURE 8 Average values of speed differential for cases with statistically significant sign type effects.**

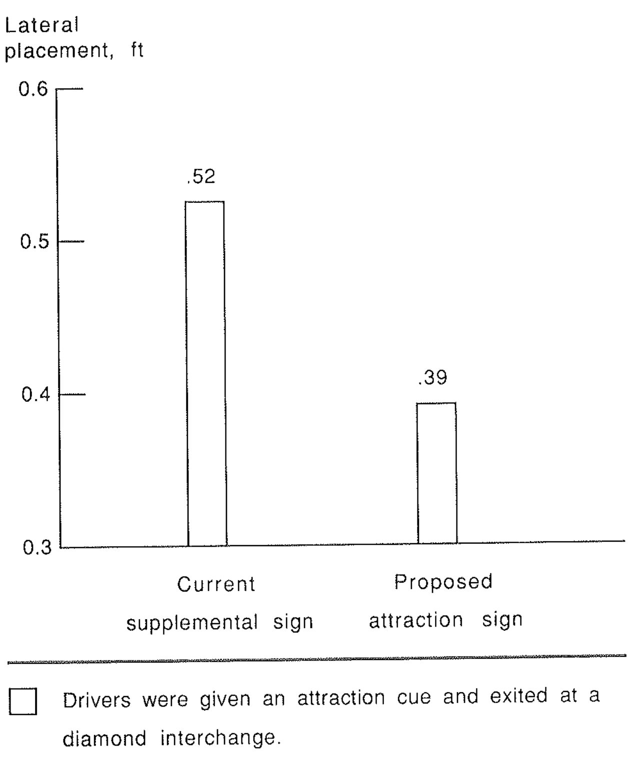


**FIGURE 9 Average values of lateral placement for the case with a statistically significant sign type effect.**

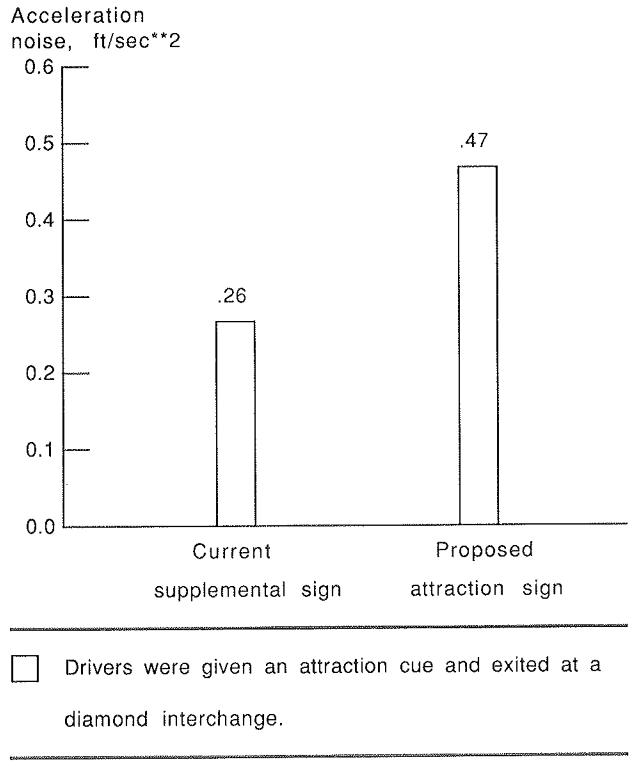


**FIGURE 10 Average values of acceleration noise for the case with a statistically significant sign type effect.**

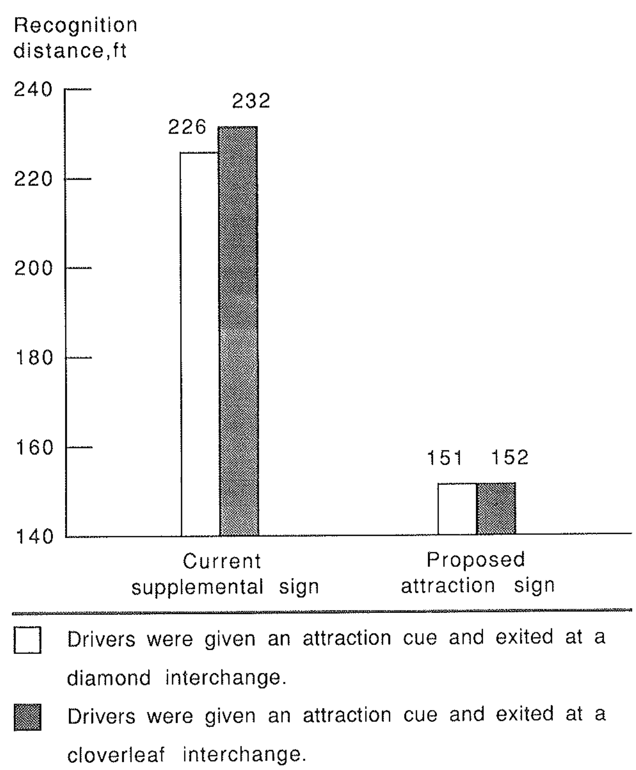


**FIGURE 11 Average values of recognition distance for cases with statistically significant sign type effects.**

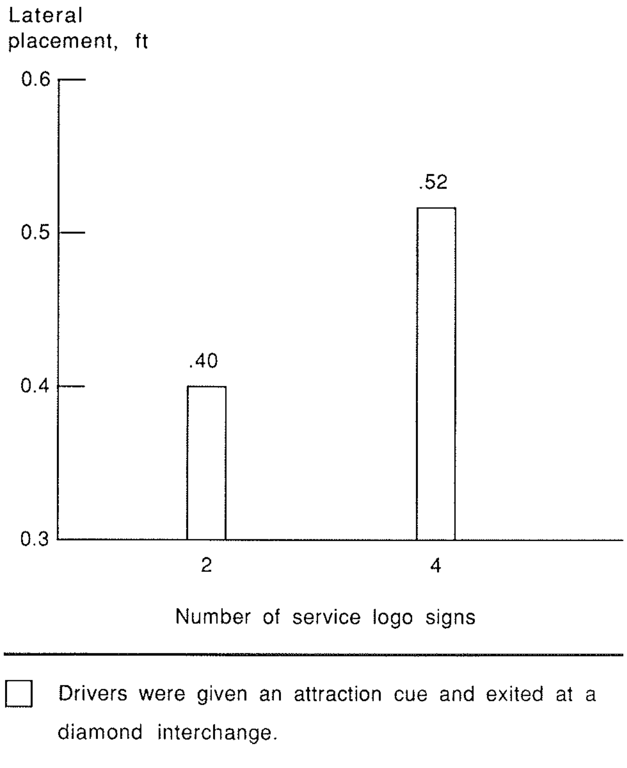


**FIGURE 12 Average values of lateral placement for the case with a statistically significant effect due to the number of service signs.**


**FIGURE 13 Average values of acceleration noise for the case with a statistically significant effect due to the number of service signs.**

of interchange geometrics and several MOEs. The proposed attraction signs on an interchange approach led consistently to poorer driver performance on the approach than the current supplemental sign on an identical interchange approach. The difference in terms of average values for MOEs was not great and the MOEs were not necessarily indicative of unsafe driving. The differences in driver performance between the proposed and current sign presentations were large enough and consistent enough, however, to cause concern should the proposed signs be installed without further study.

The design of the proposed attraction sign, rather than the sign concept, may have led to the changes in driver performance. Driver preferences for blue over brown attraction signs and better performance when scanning service logo signs with picture logos as opposed to attraction signs with word messages are indications of the need for more refinement in sign design. If picture logos are used, caution should be exercised that the pictures or symbols for local attractions are recognizable and do not further confuse unfamiliar motorists. A large letter size for the attractions may also be necessary to increase the average recognition distances.

Driver age, driver sex, and the number of service signs should be considered in the design of a future field study. All three variables were found to cause a statistically significant difference in performance under several sets of circumstances. Older drivers, female drivers, and four service signs were the levels of these variables at which the greatest differences were seen.

Several factors and MOEs were included in this study that do not need to be examined in the field. There was no significant difference demonstrated in driver performance between the different levels of the "number of logos per sign" variable for any MOE. The findings of this study should not be generalized for cases of greater than four logos per sign, because four or less messages per panel were tested. The cloverleaf interchange results differed only slightly from the results at diamond interchanges. This slight difference may be due to the smaller sample sizes available for the cloverleaf interchange analyses. Thus there may be justification for omitting this more complex geometry and sign type from a future study. The erratic maneuvers MOE appears warranted only when a very large sample size is available. The lateral placement MOE yielded anomalous data and is not recommended for use in a field test.

Two issues that were covered briefly in this study should be considered in the formulation of a field test. The sign density or sign spacing issue should not be neglected in a field test. Also, the possibility of violated driver expectancies due to inconsistent supplemental signing should be examined.

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# Factors Affecting Speed Variance and Its Influence on Accidents

NICHOLAS J. GARBER AND RAVI GADIRAJU

**One of the major factors that should be considered in selecting the speed limit for a stretch of highway is safety. It is generally accepted that the level of safety depends on certain characteristics of the traffic stream and the geometric alignment of the highway. However, in many cases speed limits are posted without adequate consideration given to these characteristics. For example, an important traffic characteristic that influences safety is speed variance. Currently, little is known about the factors that affect the variance of vehicle speeds in a traffic stream. Presented in this paper are the results of a study that investigated the influence of different traffic engineering factors on speed variance and quantified the relationship between speed variance and accident rates. A major influence on speed variance is the difference between the design speed and the posted speed limit. It was determined that speed variance will approach minimum values if the posted speed limit is between 5 and 10 mph lower than the design speed. Outside this range, speed variance increases with an increasing difference between the design speed and the posted speed limit. It was also found that drivers tend to drive at increasing speeds as the roadway geometric characteristics improve, regardless of the posted speed limit, and that accident rates do not necessarily increase with an increase in average speed but do increase with an increase in speed variance.**

Highway safety is a vital concern of transportation engineers. Research and experience have shown that highway safety can be improved by implementing countermeasures in one or more of three areas: the vehicle, the driver, and the roadway.

Countermeasures to improve the safety of the vehicle include seatbelts, collapsible steering columns, and regular vehicle inspections. Some familiar countermeasures taken in the area of the driver include education, strict licensing procedures, and alcohol regulations. Countermeasures relating to the roadway include regulatory and warning signs, guardrails, breakaway signs and lighting supports, bridge and curve widenings, speed zoning, and various construction techniques.

A traffic characteristic that relates to both the driver and the roadway is speed. Although there have been studies relating accident rates with different speed characteristics, few of these are recent and varying results have been obtained with respect to the effect of speed on accident rates. Some related results are summarized under the following:

- Speed control,
- Accident rates and speed,
- Accident rates and speed variance, and
- Influence of geometric characteristics on speed.

School of Engineering and Applied Science, Department of Civil Engineering, University of Virginia, Charlottesville, Va. 22901.

## SPEED CONTROL

Speed control is one of the most important tools to reduce speed-related accidents. However, speed control is a difficult and controversial issue, because criteria for establishing speed limits do not have the same degree of acceptability as do other traffic control tools such as no-passing zones or traffic signals. McMonagle (*1*), in one of the earliest studies on speed, stated, "It must be provided for and protected."

What speed is safe? Accidents occur at all speeds. Higher speeds may increase the chances of exposure to dangerous situations, and the rapidity at which these develop may reduce the ability of a driver to react properly and may lead to more accidents. The primary responsibility of traffic engineers is to identify safe speeds to reduce the probability of accidents occurring to a minimum. Only a few studies have developed recommendations for safe speeds on different highways. However, most of the recommendations were based on policy assessment or legislative requirements rather than traffic and geometric factors.

## ACCIDENT RATES AND SPEED

Although it is assumed generally that speed is the greatest contributing cause to accidents, some studies have indicated that this may not be true. One investigation (*2*) concluded that speed is not necessarily an important cause of accidents, but it is an important determinant of severity. A study in Minnesota (*3*) considered 40,000 accidents in which data on speeds of vehicles involved were available. Nearly 75 percent of the accidents involved some violation other than speed. If every accident in which speed was the only violation could have been prevented, the number of accidents would have been reduced by less than 10 percent. A research study in Pennsylvania (*4*) found that speeds of drivers with accident records were only slightly higher than those for drivers with no accident records.

## ACCIDENT RATES AND SPEED VARIANCE

Most research results show that higher speed variance is usually associated with higher accident rates. For example, Pisarski (*5*) pointed out that there is a significant statistical relationship between speed variance and accident rate. A study in Canada on speed and accidents (*2*) also found that speed variance may be an important factor in accidents. Cerrelli (*6*) concluded that accident rate increased as the speed of the vehicle deviated from the average speed of the traffic. A graph of accident rates by speed resulted in a U-shaped curve having the lowest value in the proximity of average speed. The risk of an accident appears to increase as vehicle speed varies from the average speed on the highway. Although these studies agree with the conclusion that speed variance significantly influences accident rates, few actually quantified the relationship between these variables.

## INFLUENCE OF GEOMETRIC CHARACTERISTICS ON SPEED

A study by Elmberg (*7*) on a newly reconstructed highway investigated the effect of the posted speed limit on the speed of drivers. The drivers paid little attention to posted speed limits and chose a speed that they considered appropriate for the prevailing conditions. This strongly suggests that geometric characteristics influence operating speeds. For example, a low posted speed limit on a highway with good geometric conditions may result in a wide range of speeds on the highway, which in turn will lead to an increase in accident rates.

This summary of research results indicates that while several speed characteristics may affect accident rates, speed variance is one of the more important characteristics. However, the factors that affect speed variance have not been studied widely. To determine the factors that significantly affect speed variance, a study was sponsored by the AAA Foundation for Traffic Safety. The main objectives were to investigate the traffic engineering factors that influence speed variance and to determine to what extent speed variance affects accident rates. Identification of these factors should help develop countermeasures that will result in minimal speed variance levels, which in turn will lead to reduced accident rates.

## METHODOLOGY AND RESULTS

### Data Collection

Appropriate sites were selected from different highway types so that representative data could be collected for each type. Test sites were located on the following types of highways:

- *Interstates:* Urban interstate, rural interstate, and freeways and expressways having the same geometric standards as those for interstates;
- *Arterials:* Urban arterials and rural arterials; and
- *Rural Major Collectors.*

Test sections were selected so that traffic volume and traffic characteristics would be constant within each section. The test sections were located between interchanges on interstates, freeways, and expressways, and between major intersections on other roads.

Candidate sites with geometric characteristics typical of the type of roads they represented were identified first for each highway type. Consideration was given to horizontal and vertical alignments, the number of lanes, lane widths, access control, land use, traffic volume, and traffic control devices.

A final set of 36 locations (given in Table 1) was then selected using the following criteria:

- Availability of adequate accident data,
- Availability of adequate exposure data,
- Ease of collecting additional data, and
- Good representation of different roads and terrains.

Data on traffic geometric characteristics and accident characteristics were collected at the selected study sites. Traffic data included hourly volumes and individual vehicle speeds, from which other statistics such as average speed and speed variance were computed. The Leupold & Stevens traffic data recorder was used to collect data on traffic characteristics.

The data compilation was based on 24 continuous hours of monitoring on weekdays (Tuesday through Friday) at each test section. The data collected were also used to determine different characteristics of their distributions, for example, skewness and kurtosis.

Since the geometric characteristics of a section of highway are represented by its design speed, design speed was used as a surrogate for geometric characteristics in this study. Design speed usually depends on the type of highway, the topography of the area in which the highway is located, and the land use of the adjacent area. The design speed for each location was obtained from the highway log sheets provided by the Virginia Department of Transportation (VDOT).

Data on accident characteristics were obtained from computerized files prepared and stored by the VDOT and the Virginia Department of Motor Vehicles. The necessary data were extracted Tuesday through Friday in each week for 1983 through 1986.

Each study site was identified by route number, city or county in which it was located, and section number. The following data were extracted for each site:

- Fatal accidents,
- Injury accidents,
- Property damage accidents, and
- Total number of accidents.

### Statistical Analyses of Data

A database was formulated suitable for use with available statistical packages. The database included the summary of accidents for 1983, 1984, 1985, and 1986 and the breakdown of accidents by type, class of highway, and traffic characteristics. This database was used to carry out the statistical analyses described below.

#### *Traffic Characteristics*

Table 2 presents a summary of the two main speed characteristics (average speed and speed variance) of the different types of highway. Although there was only a minimal difference in the posted speed limits for the different categories of roads, the average speed was much higher on interstate high-

TABLE 1 STUDY SITES

| ROUTE | CITY or COUNTY | LOCATION FROM | LOCATION TO |
|---|---|---|---|
| | | URBAN INTERSTATE | |
| 581 | ROANOKE* | RT 101 EBL | RT 116 & 460 EB |
| 95 | HENRICO** | M RT 301 SB | RT 73 WBL |
| 195 | RICHMOND* | RT 147 | RT 6 |
| 564 | NORFOLK* | RT 460 WBL | RT 337 |
| 64 | VA.BEACH* | INDIAN RIV RD | ECL CHESAPEK |
| 95 | FAIRFAX* | RT 613 | RT 241 |
| | | RURAL INTERSTATE | |
| 77 | CARROLL** | RT 69 WBL | RT 52 |
| 64 | YORK** | RT 199 EBL | W CONN RT 143 |
| 95 | PRINCE WILLM** | RT 619 | RT 234 NB |
| 66 | FAUQUIER** | E RT 175 NB | RT 245 NBL |
| 64 | LOUISA** | RT 15 NBL | RT 208 |
| 81 | ROCKBRID** | S RT 11 | M RT 11 |
| 64 | ROCKBRID** | RT 780 | RT 623 |
| | | FREEWAYS AND EXPRESSWAYS | |
| 23 | SCOTT** | RT 65 | N RT 23 BUS |
| 150 | CHESTERFIELD** | RT 360 WBL | RT 60 WBL |
| | | RURAL ARTERIALS | |
| 80 | RUSSEL** | BUCHANAN CL | NCL HONAKER |
| 58 | PITTSYLVANIA** | HALIFAX CL | RT 729 |
| 360 | AMELIA** | M RT 360 BUS | W RT 360 BUS |
| 13 | ACCOMACK** | NCL KELLER | S RT 180 |
| 10 | Surry** | W RT 31 | RT 40 |
| 17 | ESSEX** | N RT 624 | NCL TAPPAHAN |
| 15 | MADISON** | CULPEPER CL | RT 230 |
| 220 | BATH** | RT 658 | ALLEGHANY CL |
| 460 | BOTETOURT** | RT 616 | B R PKWY OP |
| 45 | CUMBERLAND** | S RT 60 | N RT 636 |
| 256 | AUGUSTA** | ROCKINGHAM CL | RT 276 |
| 29 | CAMPBELL** | RT 24 | RT 678 |
| | | URBAN ARTERIALS | |
| 360 | HANOVER** | RT 156 | W RT 360 BUS |
| 7 | FAIRFAX* | RT 123 SBL | RT 193 |
| | | RURAL MAJOR COLLECTORS | |
| 42 | BLAND** | RT 738 | E RT 52 |
| 56 | NELSON** | SE JAMESR BR | E RT 639 |
| 156 | HENRICO** | RT 60 | RT 5 |
| 301 | GREENSVILLE** | SCL EMPORIA | RT 629 |
| 55 | FAUQUIER** | W RT 17 | WARREN CL |
| 42 | SHENANDOAH** | S RT 675 | N RT 263 |
| 201 | LANCASTER** | RT 3 | N RT 600 |

* City
** County

TABLE 2 TRAFFIC CHARACTERISTICS

| HIGHWAY TYPE | AVERAGE SPEED | SPEED VARIANCE |
|---|---|---|
| Interstate | | |
| Urban Interstate | 55.73 | 73.68 |
| Rural Interstate | 57.60 | 36.75 |
| Expressway and Freeways | 52.79 | 50.02 |
| Arterials | | |
| Urban Arterials | 53.92 | 49.02 |
| Rural Arterials | 51.82 | 62.23 |
| Rural Collectors | 44.69 | 73.06 |

TABLE 3 RESULTS OF ANOVA ON TRAFFIC CHARACTERISTICS

| | AVERAGE SPEED | | | SPEED VARIANCE | | |
|---|---|---|---|---|---|---|
| VARIABLE | Computed F value | F value at 0.05 Significance | Result | Computed F value | F value at 0.05 Significance | Result |
| Average Speed | Not Applicable | | | 7.04 | 2.03 | significant |
| Speed Variance | | | | Not Applicable | | |
| Design Speed | 13.61 | 3.05 | significant | 2.42 | 2.29 | significant |
| Highway Type | 20.98 | 2.23 | significant | 5.62 | 2.29 | significant |
| Time (By Year) | 0.22 | 2.68 | not significant | 0.65 | 2.68 | not significant |
| Traffic Volume | 2.89 | 3.65 | not significant | 1.67 | 2.12 | not significant |

ways. Analysis of variance (ANOVA) was performed on the main variables, speed variance, and average speeds to test the extent to which different variables affect speed characteristics. The class variables were highway type, design speed, and time (by year). The effect of average speed on speed variance was determined by segmenting the average speeds into 3-mph ranges and performing the one-way ANOVA test. The results are given in Table 3.

The ANOVA tests confirmed that highway type has a significant effect on average speed and speed variance at the 5 percent significance level. Design speed (a surrogate for highway geometric characteristics) also has a significant influence on these variables. Both average speed and speed variance were not affected by time (year in which data were obtained). Another result obtained was that average speed affects variance. It is clear that these variables are interrelated and do not have independent influences on speed characteristics.

### *Accident Characteristics*

Table 4 presents a summary of the total and fatal accident rates on the different types of highways. The results indicate that accident rates are much lower on interstate highways, although it was shown previously that speeds were much higher on these highways. The ANOVA test was performed to test the extent to which different factors affect total accident rates. The results are given in Table 5.

TABLE 4 ACCIDENT CHARACTERISTICS

| HIGHWAY TYPE | TOTAL ACCIDENT RATE[1] | FATAL ACCIDENT RATE[2] |
|---|---|---|
| Interstate | | |
| Urban Interstate | 68.0 | 5.0 |
| Rural Interstate | 52.0 | 2.0 |
| Expressways and Freeways | 97.0 | 4.0 |
| Arterials | | |
| Urban Arterials | 230.0 | 13.0 |
| Rural Arterials | 141.0 | 4.0 |
| Rural Collectors | 169.0 | 2.0 |

1. Number of accidents per 100 million vehicle miles of travel.

2. Number of fatal accidents per 100 million vehicle miles of travel.

TABLE 5 RESULTS OF ANOVA ON TOTAL ACCIDENT RATES

| | TOTAL ACCIDENT RATE | | |
|---|---|---|---|
| VARIABLE | Computed F value | F value at 0.05 | Result |
| Average Speed | 4.46 | 2.02 | significant |
| Speed Variance | 2.35 | 1.84 | significant |
| Design Speed | 5.13 | 2.29 | significant |
| Highway Type | 8.22 | 2.29 | significant |
| Time (By Year) | 1.06 | 2.68 | not significant |

The results indicate that each of the variables (average speed, speed variance, design speed, and highway type) has a significant effect on accident rates. It should be noted, however, that there is some correlation between design speed and average speed, and average speed and speed variance. Therefore, it cannot be concluded that each of these variables independently affects accident rates.

## Model Development

The results of the ANOVA indicated that the type of highway had some impact on speed and accident characteristics. The results also indicated that for all types of highways a statistically significant difference existed between the speed variance for different categories of average speeds and between accident rates for different speed variances. Mathematical models were developed using regression analysis to quantify these observations. The models obtained are discussed below using appropriate figures. The curves shown in the figures are graphic representations of the mathematical models obtained.

### *Average Speed and Design Speed*

The average speed at each site was plotted against the design speed to indicate the general mathematical relationship. Figure 1 shows these plots. The regression analysis indicates that

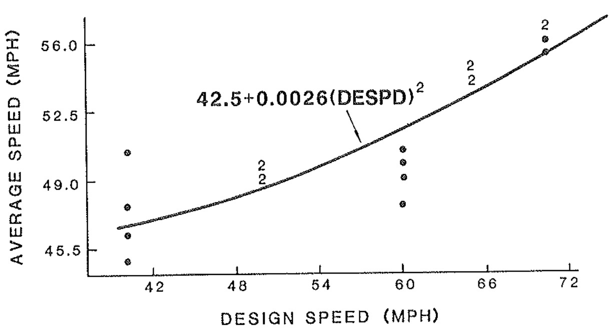


**FIGURE 1 Average speed versus design speed.**

the relationship between average speed and design speed can be given as

$$AVSPD = 42.5 + 0.0026\ (DESPD)^2 \tag{1}$$

where $AVSPD$ is average speed in miles per hour and $DESPD$ is design speed in miles per hour (40 mph $< DESPD <$ 70 mph). The coefficient of determination for this expression is 84.3 percent.

Since design speed is a surrogate for roadway geometrics and a higher design speed indicates better geometric characteristics, this model suggests that drivers tend to travel at higher speeds on highways with better geometric characteristics regardless of the posted speed limit. (All of the study sites considered for this model had a posted speed limit of 55 mph.)

### *Speed Variance and Average Speed*

Figure 2 shows plots of speed variance and average speed for all highway types. It can be seen that speed variance decreases as average speed increases. However, the relationship is nonlinear and resembles a second-order function tapering off to a constant value. This is realistic because speed variance can never go below a certain value even at higher average speeds. The relationship obtained from the regression analysis is given as

$$SPVA = -16.7 + 204803\ (AVSPD)^{-2} \tag{2}$$

where $SPVA$ is speed variance and $AVSPD$ is average speed (25 mph $< AVSPD <$ 70 mph). Results of the regression analysis show a coefficient of determination of 94 percent for this model.

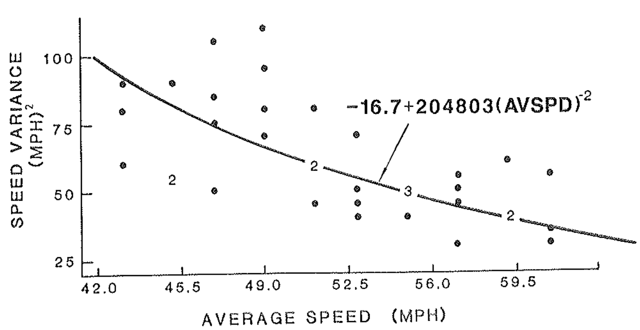


**FIGURE 2 Speed variance versus average speed.**

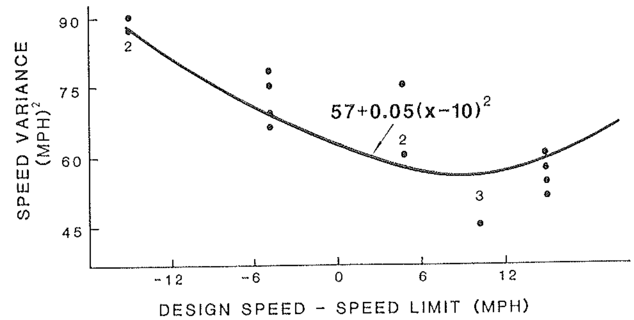


**FIGURE 3 Speed variance versus design speed minus speed limit.**

### *Speed Variance, Design Speed, and Posted Speed Limit*

The results of analyses presented earlier indicated that average speed depends on the design speed and that speed variance depends on the average speed. This suggests that the design speed has some effect on speed variance. Because another study showed that average speed at a given location is affected by the posted speed limit, a model relating speed variance with the design speed and the posted speed limit was developed. The independent variable selected for this model was the difference between the design speed and the posted speed limit. This effectively takes into account the main factors influencing average speed. These include the type of highway and geometric characteristics that are represented by the design speed and regulation, which is given in terms of the posted speed limit. The plots of speed variance against the difference between design speed and posted speed limit are shown in Figure 3. It can be seen from these plots that speed variance tends to be low when the difference is between 6 and 12 mph. In the application of this model, the range of this difference should be considered to be between 5 and 10 mph because speed limits and design speeds are usually multiples of 5 mph. The model obtained from regression analysis is given as

$$SPVA = 57 + 0.05\ (X - 10)^2 \tag{3}$$

where $SPVA$ is speed variance and $X$ is design speed minus posted speed limit in miles per hour. Table 6 presents computed values for speed variance using Equation 3.

This model suggests that minimum speed variance will occur when the difference between the design speed and the posted speed limit is 10 mph. Results of the regression analysis show that the model explains about 85 percent of the variation observed.

### *Accident Rates and Average Speed*

An attempt was made unsuccessfully to correlate accident rates with average speed for the different types of highways. Plots of accident rates against average speeds were very scattered. This indicates that there is no strong correlation between accident rates and average speed for any given type of highway based on the data used. This tends to support the theory that higher speeds do not necessarily result in higher accident rates.

When the data for all sites were pooled together and accident rates at the locations were plotted against the corre-

TABLE 6 SPEED VARIANCE VERSUS DESIGN SPEED MINUS SPEED LIMIT

| Design Speed - Speed Limit X | Speed Variance Y |
|---|---|
| - 15 | 88.25 |
| - 10 | 77.0 |
| - 5 | 68.25 |
| 0 | 62.0 |
| 5 | 58.25 |
| 10 | 57.0 |
| 15 | 58.25 |
| 20 | 62.0 |

(Equation: $Y = 57 + 0.05(X - 10)^2$)

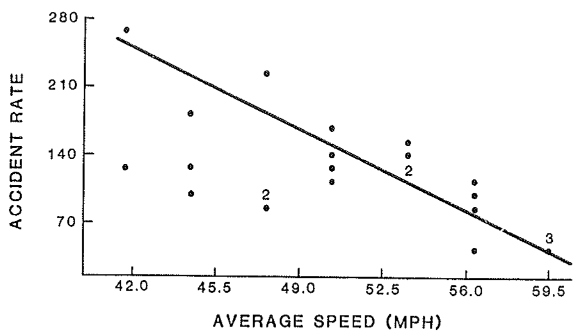


**FIGURE 4 Accident rate versus average speed.**

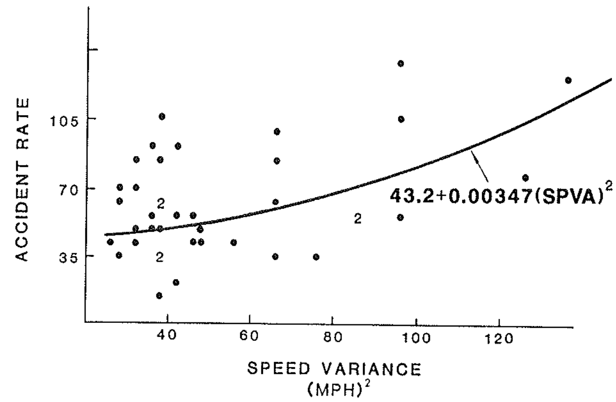


**FIGURE 5 Accident rate versus speed variance for Interstate highways.**

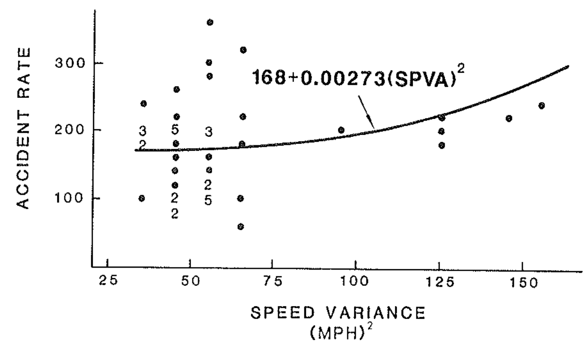


**FIGURE 6 Accident rate versus speed variance for arterial highways.**

sponding average speeds observed, accident rates decreased with increased average speeds (see Figure 4). However, it would be inaccurate to make such a conclusion because average speeds on interstate highways tend to be higher than those on primary highways, and accident rates are lower on interstate highways because of better geometric characteristics. Therefore, Figure 4 shows the effect of the different geometric characteristics rather than the effect of speed. This also explains the result of the ANOVA test recorded earlier, which indicated that average speeds significantly affect accident rates.

### *Accident Rates and Speed Variance*

Models were formulated to examine the influence of speed variance on accident rates on different categories of highways. Figure 5 shows plots of accident rates against speed variance for interstate highways.

These plots indicate clearly that accident rates increase as speed variance increases. The model obtained from the regression analysis for interstate highways describes about 70 percent of the variation observed, and is given as

$$ACCRT = 43.2 + 0.00347\,(SPVA)^2 \tag{4}$$

where $ACCRT$ is accident rate in number of accidents per 100 million vehicle miles of travel and $SPVA$ is speed variance.

The corresponding plots for arterial highways are shown in Figure 6. These plots also indicate that as speed variance increases accident rates also increase. The model explains about 82 percent of the variation and is given as

$$ACCRT = 168 + 0.00273\,(SPVA)^2 \tag{5}$$

## TEST OF MODELS

To test the validity of the models, a detailed analysis was carried out to identify sections of highways that have significantly higher accident rates than the critical values for their specific highway type. The following equation was used to determine critical accident rates for a given section of a highway:

$$C = A + K(A/M)^{1/2} + \frac{1}{2M} \tag{6}$$

where

$C$ = critical accident rate,
$A$ = average accident rate for the category of highway being tested,
$M$ = average vehicle exposure for the study period at the location (million vehicle miles),
$K$ = a constant, the $Z$-value for 95 percent confidence (1.96).

Sites with accident rates significantly higher than the critical value usually are considered hazardous locations.

Traffic and accident data files from the four study years were sorted by highway category and mean accident values were computed for each category. The critical accident value for each location then was computed. Sites at which accident rates were higher than the corresponding critical values were identified and 31 sites were selected randomly for testing the models. This resulted in 124 observations for the 4-yr study period. The difference between the design speed and the posted speed limit for each site was compared with the range of 5 to 10 mph, which the model suggests for minimum accident rates, using the following hypothesis: Speed variance will be higher on either side of a desirable zone of 5 to 10 mph for the difference between the design speed and the posted speed limit. This, in turn, will result in higher accident rates. This hypothesis can be restated as follows: If the location is hazardous, in the sense that its accident rate is higher than the critical value, then the difference between the design speed and the posted speed limit is either less than 5 or greater than 10 mph. This can be mathematically stated as follows: if $ACCR > CRIT$, $x < 5$, $x > 10$, then the hypothesis is true; if $ACCR < CRIT$, $x > 5$, $x < 10$, then the hypothesis is true; otherwise, the hypothesis is false. The test was applied to the 124 observations. The results validated the hypothesis. About 80 percent of the observations satisfied the conditions of the hypothesis.

The next hypothesis tested was: Accident rates at hazardous sites can be reduced by selecting an appropriate posted speed limit at those sites. In testing this hypothesis, appropriate speed limits within the desirable zone were selected for different sites based on their design speeds. The resulting speed variance was computed using Equation 3. The expected accident rates were computed for the respective sites using either Equation 4 or 5. Accident rates were reduced at 73 percent of the sites considered, which supports the hypothesis. The results indicate that at a relatively small percentage of the sites the hypothesis was not confirmed. However, the hypothesis was substantiated at a significantly larger percentage of the sites. These results suggest that the models given in Equations 3, 4, and 5 reasonably describe the respective relationships.

## CONCLUSIONS AND RECOMMENDATIONS

### Conclusions

The following conclusions are made based on the results of the study:

- Accident rates increase with increasing speed variance for all classes of roads.
- Speed variance on a highway segment tends to be at a minimum when the difference between the design speed and the posted speed limit is between 5 and 10 mph.
- For average speeds between 25 mph and 70 mph, speed variance decreases with increasing average speed.
- The difference between the design speed and the posted speed limit has a statistically significant effect on the speed variance.
- Drivers tend to drive at increasing speeds as the roadway geometric characteristics improve, regardless of the posted speed limit.
- The accident rate on a highway does not necessarily increase with an increase in average speed.

### Recommendations

In order to reduce speed-related accidents, speed limits should be posted for different design speeds as follows:

| *Design Speed (mph)* | *Posted Speed Limit (mph)* |
|---|---|
| 70 | 60 or 65 |
| 60 | 50 or 55 |
| 50 | 40 or 45 |

## ACKNOWLEDGMENTS

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# Measuring Road User Behavior with an Instrumented Car and an Outside-the-Vehicle Video Observation Technique

RICHARD VAN DER HORST AND HANS GODTHELP

**The limitations of human information processing capabilities are apparent in modern road traffic. Research methods for studying road user behavior range from unobtrusive observations of actual traffic to highly controlled laboratory experiments. Often the incompatibility between controllability and validity makes the choice of a suitable method difficult. Described in this paper are recent developments in two research methods used at the TNO Institute for Perception. In-car measurements using an instrumented car enable a detailed analysis of the behavior of the individual driver. The behavior of arbitrary road users can be analyzed quantitatively by outside measurements with video techniques. Recently developed computer-based techniques allow the quick, efficient, and flexible use of both methods. The methods are illustrated by examples of recent research in which driver behavior is analyzed in terms of time-related measures. At the control level of the driving task, several results can be explained by the Time-to-Line-Crossing (TLC) measure. At the guidance level, the Time-to-Collision (TTC) measure defines the severity of interactions between road users. The correspondence between the results of both approaches suggests that drivers directly use such time measures for vehicle control and decisionmaking strategies in traffic.**

Traffic safety problems are often caused by limited human capabilities in reacting to a wide range of traffic situations. With higher speeds and increasing traffic densities, the road user cannot evaluate all information correctly. Therefore, it is important to become familiar with the functional limitations of the individual road user.

To collect basic knowledge on driver information processing and to translate this knowledge into optimal roadway and vehicle ergonomics, a wide variety of research methods is necessary. These methods differ on the controllability of relevant aspects or circumstances, the unit of measurement, the selection of measurable aspects, and the validity of the results for traffic safety. Incompatibility between controllability and validity exists in most traffic safety research.

The ultimate way to study road user behavior is to analyze accident data. Although accident data are a direct measure of traffic safety, they are concerned only with the outcome of the traffic system. Furthermore, the recording of accident data is usually incomplete, difficult to access, and often not usable in evaluation studies because of the problem of small numbers. A major problem is also that suitable control data such as exposure measures and characteristics of the task environment are seldom available.

Some of the methodological shortcomings of accident data analysis may be overcome with well-controlled laboratory research methods. Driving simulators are a valuable tool for specific research questions. The required completeness of a driving simulator facility depends on the nature of the research question. In many cases, a part-task simulation may be sufficient. For example, in measuring threshold values for the perception of lateral speed, the total dynamic visual road scene has to be presented but subjects do not have to actually steer a vehicle. They only have to decide on a non-zero value for that variable. Such part-task simulations are useful for analyzing the driving process in detail. However, generalization of findings to normal driving behavior is often difficult.

The lack of reliability of accident analysis data and the limited validity of laboratory simulation makes the development of real-world observation techniques relevant for road user behavior research. At the TNO Institute for Perception two techniques have been refined recently, an instrumented car and a video observation and analysis technique. Modern technologies enable a quick, efficient, and flexible use of these research methods. Each of them will be described in this paper. In addition, the use of both techniques to enlarge our understanding of the strategies adopted by drivers in actual traffic will be illustrated by an example of recent research in which behavior was analyzed in terms of time-related measures.

## INSTRUMENTED CAR FOR ROAD USER STUDIES (ICARUS)

The use of instrumented cars as a "general purpose" driving laboratory for road user studies has increased gradually since 1960. At that time, conventional electronics and tape recorders were used to meet basic data monitoring and storage requirements (*1*). In recent years, the development of microprocessor and microcomputer technology stimulated the use of flexible data acquisition systems in instrumented cars (*2,3*). The major advantage of the equipment used in the TNO instrumented car ICARUS is that it provides quick and standardized pro-

TNO Institute for Perception, P.O. Box 23, 3769 ZG Soesterberg, The Netherlands.

TABLE 1 SOME SPECIFICATIONS OF VARIABLES AS MEASURED IN THE INSTRUMENTED CAR ICARUS

| Variable | Sensor | Manufact. | Model | Range | Accuracy |
|---|---|---|---|---|---|
| steering-wheel angle | potentio-meter | Bourns | 6574-1-103 | $\pm 45^o/270^o$ | ±0.1% |
| yaw velocity | gyro | Notronic | 76506/ D 1963551 | $\pm 100^o$/s | ± 1 % |
| vertical accel. | strain gages | Statham Instruments | A5-2.5-350 | ±2.5 g | ±10 %* |
| lateral " | " | " | A4- 1 -350 | ± 1 g | ±10 %* |
| longitudinal " | " | " | " | " | " * |
| driving speed | Hall transducer | Siemens | FP211D155 | 0-144 km/h | 0.16 km/h (10 Hz) |
| lateral position | see text | TNO | | -1 to +2.5m | ± 1 % |

* unfiltered

cedures to set up and execute experiments. An overview of the variables measured and the system configuration is presented below. While in the ICARUS, the subject is accompanied by an experimenter and a technician. The technician takes care of the equipment and the experimenter gives instructions to the driver and supervises the experimental procedure. When working with inexperienced subjects or in risky situations, a driver-training instructor acts as experimenter who can take over with dual controls.

## Measurement of Variables

The variables of the driving process can be grouped into different categories, that is, driver inputs, driver outputs, physiological parameters, and vehicle motion characteristics. Table 1 presents the technical specifications and details about manufacturer and type of sensor for the major variables.

### *Driver Input*

The variables in this category include all input information available to the driver and describe the relevant aspects of the environment, that is, preview, road geometry, road signs, and interacting traffic. Driver input consists also of specific variables related to the movements of the vehicle.

Driver visual input is difficult to record. Although the view in front of the car can be recorded easily with video equipment, it is difficult to gain further knowledge about the driver's information selection process. In earlier versions of the ICARUS much effort was spent to study the driver's information selection by adding the driver's head and eye movements to the video picture (*4*). In the mid-1970s, however, interpretation and data analysis problems with eye-movement results made us decide to reduce this type of research. Cognitive-oriented research on visual selection processes, driver attention, and in-car displays may stimulate a renewed interest in eye-movement analysis (*5,6*). Time-domain analyses of looking strategies can also be made by using occlusion techniques. The ICARUS is provided with a spectacles-mounted liquid-crystal occlusion device PLATO (Portable Liquid-crystal Apparatus for Tachistoscopic Occlusion) with open and closure times of less than 3 ms. Milgram (*7*) gives a detailed description of this device and an example of its use will be presented later.

A programmable electro-luminescent display (Deeco, M3EL 512 × 256 PS) and voice inputs are available to analyze the influence of existing and future control and communication aids inside the car. Information on driver visual input can also be gathered by the experimenter on a pushbutton unit for labeling special events in the driving environment such as overtaking maneuvers, interacting traffic, or the presence of road signs.

In some experiments, driver performance and workload are measured by introducing an additional task in which the driver has to react to specific auditory or visual stimuli.

### *Driver Output*

As a result of the perceived information, the driver may anticipate and react to the traffic situation by changing course or acceleration. These actions also have to be recorded. Steering wheel movements are measured by a potentiometer attached to the steering column (for specifications see Table 1). Pedal actions are measured with potentiometers and switches, gear-lever position, and all other controls with switches. The horn-lever switch can be used for driver response.

### Vehicle Motion Characteristics

The driver's actions define performance in terms of vehicle motions, accelerations, velocities, and road positions.

The three translational accelerations are measured with three vehicle-fixed one-dimensional accelerators mounted just behind the driver.

Forward velocity and distance traveled are measured with a pulse-counter and Hall transducer mounted on the drive shaft. The rotational velocities around three axes are measured with small gyros.

The lateral position of the vehicle in relation to the road geometry describes lateral vehicle control on straight and curved roads. The ICARUS is equipped with a special device developed by the Institute. It continuously measures the distance between the vehicle and a reference line. Any sufficient change of contrast at the side of the road can be taken as reference. The transducer is mounted at the back or beside the ICARUS and scans the intensity of reflected light in a lateral plane across the road through a fast rotating prism in front of a photoamplifier. Contrasting light levels (reference line) cause peaks in the scanning signal, which are used to measure the prism rotation angle at that specific moment in relation to the vertical position. A tangent-transformation gives the lateral distance to the reference line. A given sector of the lateral plane can be selected by an electronic adjustable scanning window to avoid incorrect measurements because of passing vehicles or other disturbing objects. The lateral position transducer operates in a wide range of ambient light. During night conditions, however, it is necessary to illuminate the road in the lateral plane by additional spotlights on the vehicle.

### Physiological Parameters

Indications of driver performance and workload may be derived from variables describing the driver's physiological state. Several physiological variables can be measured, for example, heart rate, respiration rate, galvanic skin response, and special features of the electroencephalogram (EEG).

## System Configuration

The variety of projects requires a system that allows flexible, efficient, and reliable registration of a given subset of variables and presents stimuli dependent on the experimental design. Therefore, the system is built around a computer with floppy disk data storage. Figure 1 shows a schematic diagram of the basic configuration. The analog to digital (A/D) input channels with a resolution of 16 bits, D/A output channels of 12 bits, and the digital channels of 24 bits each cover a given set of variables. A special interface unit allows the use of all kinds of analog input variables.

A set of batteries, a 24 V/DC generator (Motorola, 8SA3005, P-10-24) attached to the engine, and two converters of 220 volts/AC, 500 watts each (Blessing Electronics, BTY 24/500/S) deliver the power supply for all the equipment. An automatic power-off safety switch prevents additional damage in case of a collision.

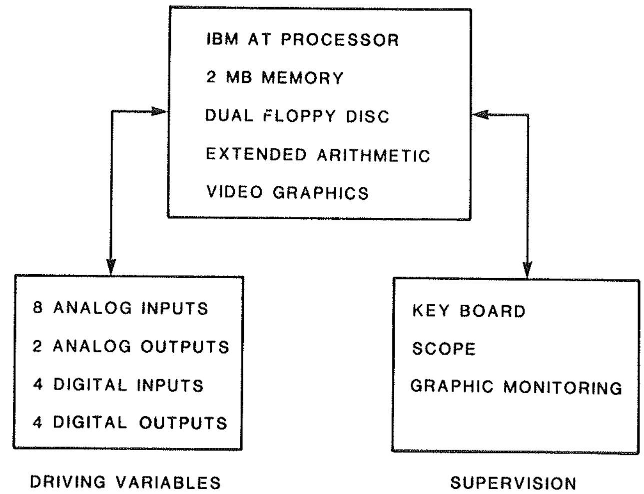


**FIGURE 1 Basic configuration of the data-management system in the instrumented car ICARUS.**

Standard procedures are used for checking, calibrating, and monitoring the analog and digital channels with the help of a graphics display in front of the technician. All data of each experimental run are stored on floppy disks in data files that also contain all information related to experimental conditions, system configuration, and calibration. A storage capacity of more than 1 Mbyte enables an unrestricted sampling time of several hours for eight channels at 10 Hz. The maximum sampling frequency depends on the number of channels and is approximately 300 Hz for eight channels or 2500 Hz for one channel.

## VIDEO ANALYSIS OF ROAD TRAFFIC SCENES (VIDARTS)

The use of an instrumented car is appropriate in studies where both the input to the driver and the driver output to the system are essential. For studying interactions between road users or between road user and environment, it is often sufficient, or even better, to observe behavior in terms of vehicle movements. These can be observed unobtrusively in actual traffic situations. Recording traffic scenes allows reliable data collection and detailed analyses of behavior.

The development of a quantitative method for analyzing road traffic scenes started with a comparison between film and video. Both techniques have specific advantages and disadvantages, but video was highly preferred with respect to cost, practical aspects, and potential for future technical developments (*8*). A semiautomatic procedure for analyzing video scenes was developed, enabling a quantification of movements of any given vehicle in actual traffic.

Van der Horst (*9*) describes an early version of the video analysis system. A recent implementation on an IBM/AT computer with a video digitizer card extended considerably the flexible and interactive use of the system.

**General Procedure**

The procedure consists of making videorecordings with one or more fixed cameras on the spot and a subsequent off-line quantitative analysis at the laboratory with a specially developed video plotting device. A fully automated real-time image processing procedure would be preferred, but such an approach is not realistic because of the current state of technology. First, a real-time analysis might be needed in video monitoring road traffic for traffic surveillance, traffic control, or traffic management. In most of our applications, however, the requirement for real-time processing is not urgent. Second, in many applications it appears to be effective to preselect relevant events before a detailed quantitative analysis. Furthermore, a process with interactive communication between the system and a human operator reduces the complexity of the analysis procedure.

The quantitative analysis consists of selecting positions of some points of a vehicle by positioning an electronic cursor on a video still. By a transformation based on at least four reference points, the $x$- and $y$-coordinates of the video plane are translated into positions on the road plane. By analyzing successive video stills, a sequence of positions over time is obtained from which other variables such as velocity, acceleration, heading angle, and time-to-collision measures are derived. Sometimes a more simple procedure can be applied, that is, measuring time moments of passing given successive road positions instead of measuring positions at successive time intervals. For example, by measuring the moments when two lines with a known distance in between are passed by a vehicle, a simple speed measurement can be obtained.

**Data Collection and Analysis**

*Videorecordings*

The first step in data collection consists of making videorecordings of traffic scenes at intersections or road sections. A suitable place for mounting the camera has to be found at the location, preferably at a height of more than 4 m above the road surface. A primary requirement is that the observations do not influence the driver behavior. For that reason, placing a video camera on the roof of a van or on a telescopic mast (*10,11*) was rejected. A good and unobtrusive camera position can be found either in an adjacent building, on a balcony, in a lamppost, or in other elements present at the location.

Basically, the videorecording equipment consists of three parts—the camera, a combined synchronization/timer/field-encoder unit, and a videorecorder. Until now, black-and-white videocameras have been used because of their superior horizontal resolution. A wide variety of fixed focal length and zoom lenses in the range from f = 5.8 to 100 mm is available to ensure an appropriate field of view (for a ⅔-in. camera between 78° and 5° horizontally). When the outlook of the location is too limited or the area is too large to be covered by one camera, additional cameras, videowiper(s), or recorders are optional. It is important that all equipment is synchronized correctly and that each video signal to be recorded separately is coded identically by the timer/field-encoder unit. This unit superimposes a numerical display of date and time of day (up to 1/50 sec) onto each video image and a special digital code at the beginning of the video lines. This 24-bit digital code (one-to-one related to the time of day) uniquely labels each video field and is used for the computer-controlled search for any given image during the analysis procedure.

Usually, the recordings are made by a Umatic videocassette recorder (speed 50 fields/sec). When no direct judgments from tapes have to be made by human observers and the frequency of occurrence of relevant events is low, a time-lapse videorecorder can reduce considerably the amount of material. A reduction factor of 4 (12.5 fields/sec) enables a full 12-hr period to be stored on one 180-min VHS videocassette. Table 2 gives the specifications of the videorecording equipment.

*Video Plotting Procedure*

The basic configuration of the video plotting device is shown in Figure 2. An IBM/AT computer (640 Kb of RAM memory, 20 Mb hard disk, 8 expansion slots) forms the central part of the system. Two expansion slots are used for two interface cards that provide up to 4 parallel input and 4 output channels of 24 bits each.

One of the laborious tasks in analyzing videotapes is the precise positioning of the tape at the right image. The videorecorder operates under full computer control. Any given picture is searched automatically by the computer by the special digital time code stored in each video field. The decoder for reading the digital video code is one of the few institute-made devices. A low-cost FOR-A time-base corrector (type FA-400PS with a correction range of 2 fields and a horizontal resolution of >320 lines) is used to enhance the sync part of the video signal before it is processed by the rest of the system. The video processor with frame grabber and an 8-bit 1024 × 1024 video RAM memory (of which 520 × 576 pixels are effectively in use) (Matrox, PIP-1024) enables a flexible use of the equipment because many features can be implemented in software. For example, for simple speed measurements, a programmable electronic grid can be generated; for measuring positions, a cursor can be manipulated either by computer or human operator control. Also, simple image processing procedures, such as automatically detecting traffic signal changes directly from the video image, can be implemented rather easily in software. A special additional keyboard with 24 programmable function keys enables the operator to communicate efficiently with the system. For the first few images of a sequence, the operator has to position the cursor on a given point of the vehicle. When a few images have been analyzed, an algorithm predicts the expected position of the point based on the $x$- and $y$-positions in previous images. Then, the operator only has to indicate small corrections on those estimated positions.

*Transformation From Video-Image Plane to Road Plane*

In earlier studies on analyzing film or videorecordings quantitatively, a grid transformation was used for the conversion of image coordinates to road coordinates (*12,13*). However, assuming that (a) all points of the road surface are in one flat

TABLE 2 SPECIFICATIONS OF VIDEORECORDING EQUIPMENT AS USED FOR VIDARTS

| Equipment | Type | Manufact. | Model | Horizontal resolution (lines) | S/N ratio (dB) | Minimum illumination (lux) |
|---|---|---|---|---|---|---|
| B/W camera | 1/2" CCD | Philips | LDH0600/00 | >450 | 50[1)] | 1.5 |
| " | 2/3" CCD | JVC | TK-S310 | >530 | 50[2)] | 2 |
| " | 2/3" vidicon | Sony | AVC 3250CE | >550[3)] | 44 | 150 |
| timer[4)] | | FOR-A | VTG 33 | | | |
| recorder | Umatic | Sony | VO 5800 PS | >340 | 48 | |
| " | " | " | VO 5850 P | >340 | 48 | |
| " | VHS | Panasonic | NV-8050-E | >300 | 45 | |
| video wiper | | Sony | CMW-110CE | | | |
| mixer/wiper | | Videomatte | VM1 | | | |

All equipment is based on the PAL video standard (interlaced 2:1, 625 lines/50 Hz).

1) at 20 lux

2) at 2000 lux

3) in center, in corners > 400 lines

4) modified version with built-in synchronisation/field encoder unit.

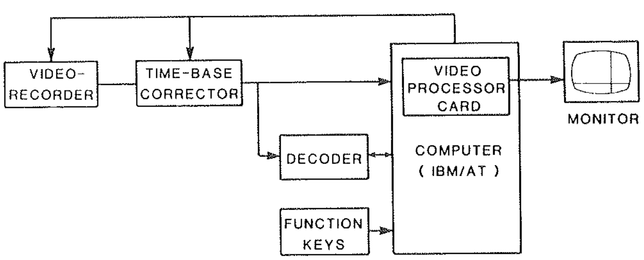


**FIGURE 2 Basic configuration of the video plotting device**

plane and (b) no distortion errors from the optics of the camera occur (correct central projection), the method of two-dimensional projective coordination (*14*) is much simpler to apply.

The transformation of video image coordinates ($X_v$, $Y_v$) to road coordinates ($X_r$, $Y_r$) is given by a broken linear function of the following form:

$$X_r = \frac{C_1 \cdot X_v + C_2 \cdot Y_v + C_3}{C_7 \cdot X_v + C_8 \cdot Y_v + 1}$$

and

$$Y_r = \frac{C_4 \cdot X_v + C_5 \cdot Y_v + C_6}{C_7 \cdot X_v + C_8 \cdot Y_v + 1} \qquad (1)$$

The coefficients $C_1$ to $C_8$ can be calculated if the coordinates of at least four points are known both in the image plane and in the ground plane. Substituting the $X_r$, $Y_r$, $X_v$, and $Y_v$ coordinates of the four points in Equation 1 results in a system of eight linear equations with $C_1$ to $C_8$ as variables. This system can be solved if no combination of three points in either plane is on a straight line. This method offers the practical advantage that nothing has to be known about the internal and external orientation of the camera. All information is included in the way the four reference points are projected on the image plane. The smaller the pitch angle of the camera the more sensitive to measuring errors the transformation will be. In general, accuracy will improve when the optical axis of the camera is as vertically oriented as possible (the higher the camera position the better) and the four reference points are spread maximally over the area to be covered.

To have a check on the transformation and to conduct an optimization procedure on the transformation, some additional reference points have to be included. A number of eight to ten points appears to give reasonable results.

In actual road scenes it will almost always be possible to find natural markings that are clearly readable from the video image.

### *Computation of Motion Parameters*

Accuracy in the measurement of positions may be influenced by many factors. A relatively high position of the camera(s), a large pitch angle, and careful selection and accurate measurement of reference points are first steps in reducing systematic errors. An optimization of the transformation coefficients based on all available reference points minimizes the influence of random errors in the measurement of the reference points. Parallax errors can be avoided by taking the contact point between tire and road surface as the precise point to be measured. By filtering the video coordinates before the transformation to the road plane, by determining the resulting position from two or three separate points of the vehicle, and by smoothing successive positions by means of a second-order polynom filtering, the overall accuracy can be improved considerably. For most applications at urban intersections, the overall accuracy for measuring vehicle position is estimated to be better than $\pm$ 0.05 to 0.1 m. Further details are presented in Van der Horst (*15*).

Vehicle velocity is obtained by differentiating successive positions with respect to time, and acceleration is found by differentiating successive velocities. For most applications it is not necessary to analyze every individual video frame; a selection of one picture out of every 12 (one picture/0.24 sec) appears to be a reasonable compromise between accuracy and duration of plotting.

## APPLICATIONS

In recent years both ICARUS and VIDARTS have been applied in a series of road user studies. Instrumented car research resulted in fundamental knowledge on course and speed perception (*16,17*) and its consequences for roadway delineation systems (*18*). Video observation research gave new insights into driver decisionmaking processes at intersections and railway grade crossings (*19–21*). VIDARTS also proved to be a useful tool to analyze the interactive decision process between bicyclists and car drivers at particular intersections (*22*).

Both techniques will be illustrated by a short description of research projects in which driver behavior is analyzed in terms of time-related measures.

### Time-to-Line-Crossing Analysis Based on ICARUS Measurements

Most of the traditional models on steering control are based on the assumption that the driver acts as a path error-correcting mechanism continually allocating attention to the steering task. However, it can be shown easily that this assumption is incorrect. Godthelp et al. (*23*) developed the Time-to-Line-Crossing (TLC) approach as a description of driving strategy that considers the driving task merely a supervisory task. With the TLC approach, predictions are made on the basis of a preview-predictor model about the time periods during which, for instance, path errors can be neglected. This prediction process assumes that (a) the vehicle starts from its momentary lateral position and heading angle and (b) the steering wheel remains fixed at its momentary position. At each moment, TLC can be calculated from the ICARUS lateral position, heading angle, speed, and commanded steering angle. Thus, the time needed by the vehicle to reach either edge of the lane is quantified. Figure 3 shows an example of a time history of these signals with the TLC measure.

In an experiment on straight lane keeping, Godthelp et al. (*23*) illustrated that TLC may serve as a valid predictor of a driver's self-chosen occlusion times for a broad range of speeds (20 to 120 km/hr). Subjects made a series of runs in the right lane of an unopened freeway with normal lane markings. Half of the runs were made with normal vision and the other runs were conducted with a voluntary occlusion. In the latter conditions, subjects wore the occlusion device as mentioned earlier. In its normal state, the occlusion device was closed, that is, the visual field was completely occluded. When the horn lever was pressed, the device switched to the "open" mode, which lasted 0.55 sec. Subjects were instructed to drive safely in the right lane and to request 0.55-sec looks whenever necessary. It was made clear to the subjects that this was not an experiment in risk taking and that wandering beyond the lane markings should be prevented. Normal precautions were taken regarding randomization of conditions, fail safe circuitry, etc.

The results of this study showed an almost perfect correlation between representative values of the TLC distribution and the occlusion times as voluntarily chosen by the subjects. This finding indicated that experienced drivers use consistent internal representations regarding the influence of speed on the duration of periods available to neglect path errors in vehicle control. Godthelp and Kaeppler (*24*) replicated this experiment with cars of different handling characteristics and found the same consistencies for understeering cars. However, in oversteering vehicles, drivers tended to accept relatively large occlusion periods despite the occurrence of very low TLC levels at the higher speeds.

The TLC approach allows us to gather more detailed information about the process of driver adaptation to vehicle characteristics. A recent study (*25*), with a procedure similar to the experiments described above, focussed on whether a driver's adaptation to road width variation, as quantified in occlusion times, could also be explained from TLC data. Again, subjects made runs at different speeds (20, 60, and 100 km/hr) on a straight highway. The major independent variable (lane width) was varied in steps of 0.5 m, giving four width levels—2.05, 2.55, 3.05, and 3.55 m. For each run, median occlusion times and 15-percent TLC levels were calculated (15 percent of the absolute TLC values in a given run is below the "15-percent level"). Table 3 gives 15-percent TLC values and occlusion times ($T_{occ}$) averaged over subjects and replicated for different speed and road width levels. For the three largest lane widths, 2.55, 3.05, and 3.55 m, the same consistent relationship between $T_{occ}$ and the 15-percent TLC values was found as in the earlier experiments. This indicates that drivers use a correct internal representation about the relation between

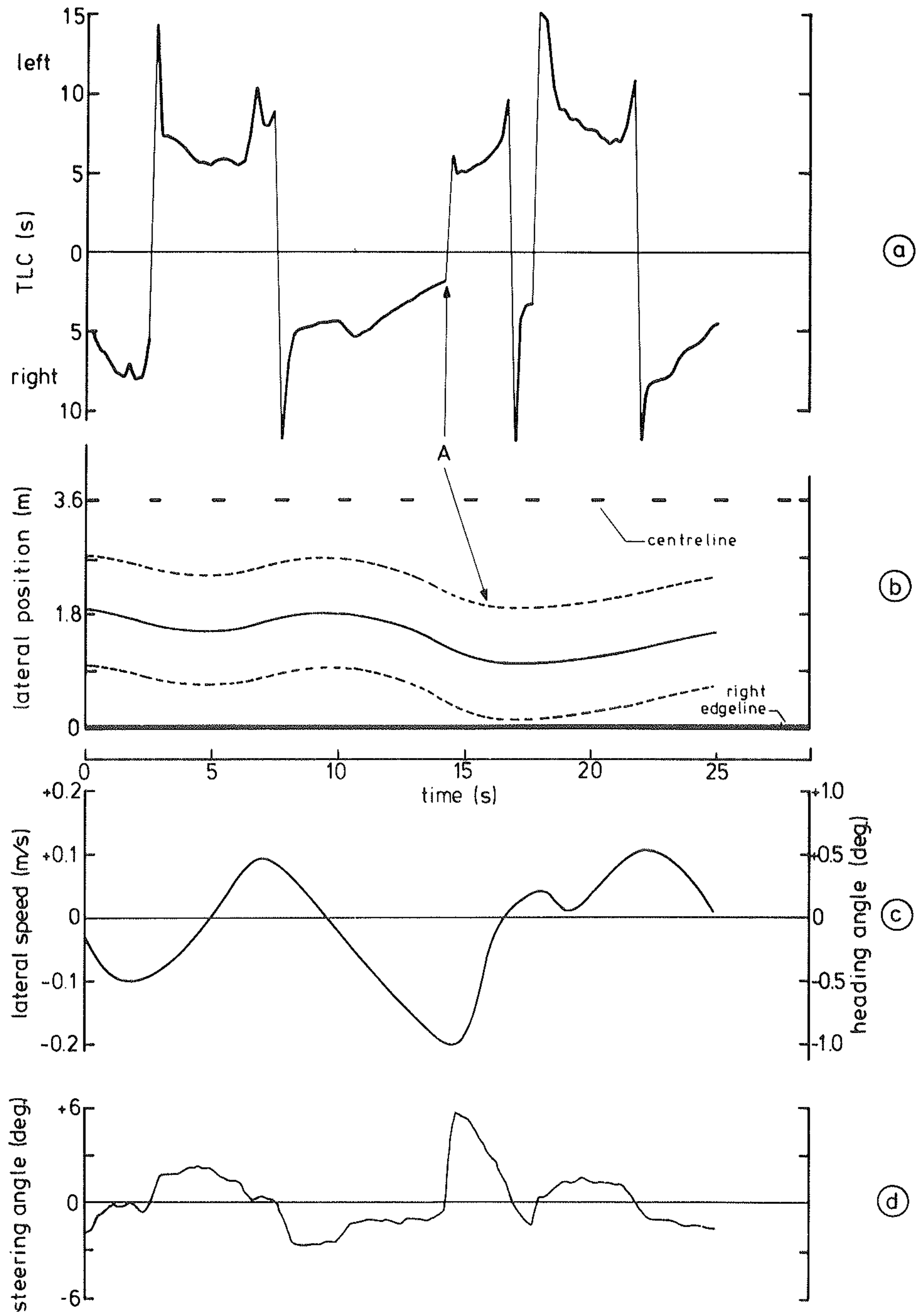


**FIGURE 3 Sampled time history for a TLC signal resulting from the corresponding lateral position, heading angle, and steering-wheel angle.**

lane width and the duration of the time available to neglect path errors. However, the 2.05 lane width data are clearly different and show that drivers tend to accept relatively large occlusion periods as compared to the 15-percent TLC values. This finding might point to a misinterpretation of the driving situation with this narrow lane. A more detailed analysis of this phenomenon is presented elsewhere (*25*).

In the occlusion experiments presented here, the TLC approach is used to quantify the potential role of visual open-loop and path-error-neglecting strategies. Basically, TLC represents the time available for a driver to neglect path errors until any part of the vehicle reaches one of the lane boundaries. The strategies adopted by drivers during error neglecting can be analyzed further by quantifying the decision rules used by drivers who switch from error neglecting to error correcting when approaching the edge of a lane. Godthelp (*26*) analyzed these rules for a straight lane-keeping task. In a condition with normal visual feedback, subjects were instructed to neglect vehicle path errors and to switch to error correcting only when the vehicle heading could be corrected comfortably to prevent crossing the lane boundary. Again, runs were made with ICARUS at three speeds (20, 60, and

TABLE 3 OCCLUSION TIMES ($T_{occ}$) AND 15 PERCENT TLC VALUES FOR THREE SPEEDS AND FOUR LANE WIDTH LEVELS

| Speed | Lane width 2.05 m | | 2.55 m | | 3.05 m | | 3.55 m | |
|---|---|---|---|---|---|---|---|---|
| (km/h) | $TLC_{15\%}$ | $T_{occ}$ | $TLC_{15\%}$ | $T_{occ}$ | $TLC_{15\%}$ | $T_{occ}$ | $TLC_{15\%}$ | $T_{occ}$ |
| 20 | 2.0 | 2.4 | 4.8 | 3.6 | 5.9 | 3.8 | 7.8 | 4.3 |
| 60 | 1.4 | 1.4 | 3.1 | 2.0 | 3.9 | 2.5 | 4.9 | 2.9 |
| 100 | 1.2 | 0.9 | 2.7 | 1.4 | 3.6 | 1.9 | 4.0 | 2.1 |

Times are given in seconds.

100 km/hr). The results showed that the lateral distance from the lane boundary at which drivers switch to error neglecting increases linearly with lateral approach speed. This mechanism results in an approximately constant TLC of 1.1 sec. This result is consistent over a broad range of speeds.

It can be concluded from this TLC analysis that an integration of variables measured with an instrumented car may enlarge our understanding of the strategies adopted by drivers in various vehicle control situations.

### Time-to-Collision Measurements Based on VIDARTS

In research on traffic safety problems at single locations and in evaluating safety measures, the information available from police accident records is of little use in analyzing the chain of events leading to an accident (*27*). The processes that result in near-accidents or serious traffic conflicts have much in common with the processes preceding actual collisions; only the final outcome is different (*28*). Techniques have been developed in several countries to systematically observe traffic conflicts using individual observers. Large differences in local circumstances result in a variety of definitions, observation methods, and severity scores. Therefore, in 1983 an international calibration study at Malmö, Sweden was conducted to compare the various traffic conflict techniques (*29*). Except for a comparison of the scores by eight different observer teams, the subjective scores could be related to several objective measures obtained by our video observation technique (*30*).

The Time-to-Collision (TTC) measure appeared to be one of the major factors in explaining a common severity dimension based on the subjective scores by the observer teams. The TTC as defined by Hayward (*12*) is the time for two vehicles to collide if they continue at their present speed and on the same path. Figure 4 shows what happens when a car approaches a fixed object. The time histories at the left represent an evasive action of normal braking; at the right, one of very hard braking. Point A indicates the TTC when the evasive action is started, representing the available maneuvering space at the moment of braking. Point B gives the minimum TTC value, reached during the approach. In more complex interactions of two moving road users, the collision course often is ended before Point B. In such cases, the TTC curve would not be concave. But even then, the minimum TTC value indicates how close an actual collision was. Both TTC values (at Points A and B) play an important role in determining the severity of a conflict. In general, only interactions with a minimum TTC less than 1.5 sec are considered critical. Trained observers are able to consistently apply this threshold value (*30*).

In an explorative study to identify and describe the rules applied by road users in a priority situation, behavior at a general rule (right-hand, right-of-way) intersection and a yield sign intersection was analyzed with VIDARTS (*20,31*). All encounters between traffic from two legs of the yield sign intersection were analyzed. The distribution of minimum TTC values showed only a few encounters with a minimum TTC less than 1.5 sec; a distinct 1.5-sec threshold value for minimum TTC appeared to be present (Figure 5). An analysis of approach profiles of free riding straight-on and left-turning motorists from the minor road revealed a similar minimum time to the intersection of 1.5 to 2 sec before deciding to proceed. The right-turning motorists did not show a minimum in their approaches.

These results suggest that time-related measures are important in describing road user behavior in actual traffic situations. Consequently, the question arises whether time measures such as the TTC are used directly by road users as a cue to decisionmaking in traffic. This hypothesis is the subject of continuing research (*32*).

The video technique appears to be an especially helpful tool for studying interactions between road users, because several dynamic elements in the approach process can be investigated in an integrated manner.

## FINAL REMARKS

As major failures in the road traffic system are eliminated, research methods to improve traffic safety and operation will

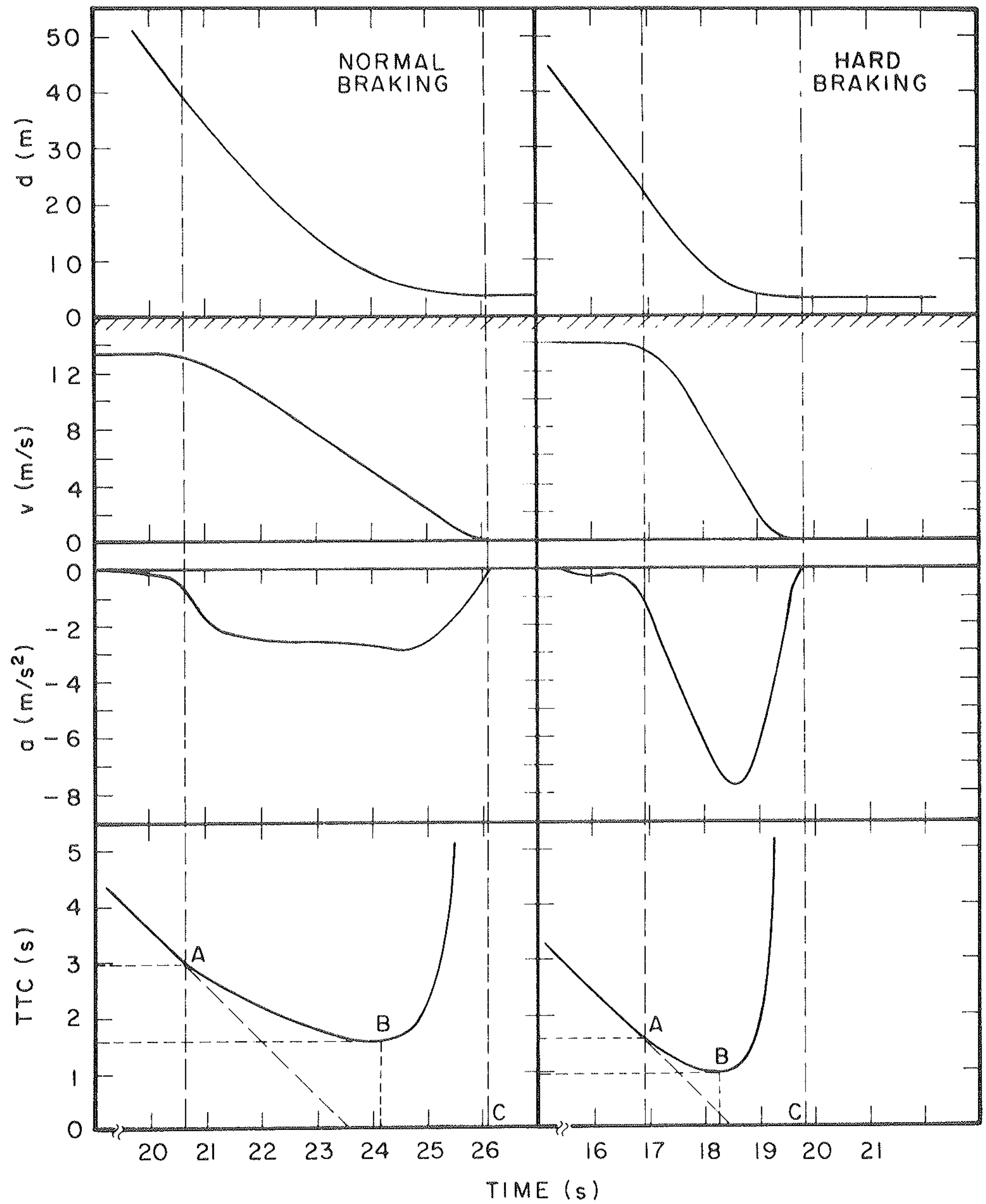


**FIGURE 4 Examples of time histories of normal braking (left) and very hard braking (right) of a car approaching a fixed object. From top to bottom: $d$ = distance to object, $v$ = velocity, $a$ = acceleration, and $TTC$ = time-to-collision.**

have to be more complex and refined. More detailed information on how the road user functions will be needed.

This paper addressed research methods for studying road user behavior with an emphasis on instrumented car and video observation techniques. The method used depends on the objectives of the specific research. The variables available by using an instrumented car such as the ICARUS enable a detailed analysis of driver behavior. The ICARUS was equipped recently with a telemetric system to study interactive behavior between road users for simple situations such as car-following and to measure behavior in a car equipped with relatively simple instrumentation.

The video observation technique has been proved to be very helpful in analyzing interactions between road users or between a road user and the environment in actual traffic. However, the work of the human operator is still time-consuming and strenuous and further automation is needed. The development of an automated system for road traffic scene analysis has made a promising start (*33*).

The correspondence in the results of the analysis of driver behavior in terms of time-related measures such as TLC and TTC suggests a fundamental relationship with driver control and decisionmaking strategies. This is the subject of continuing research.

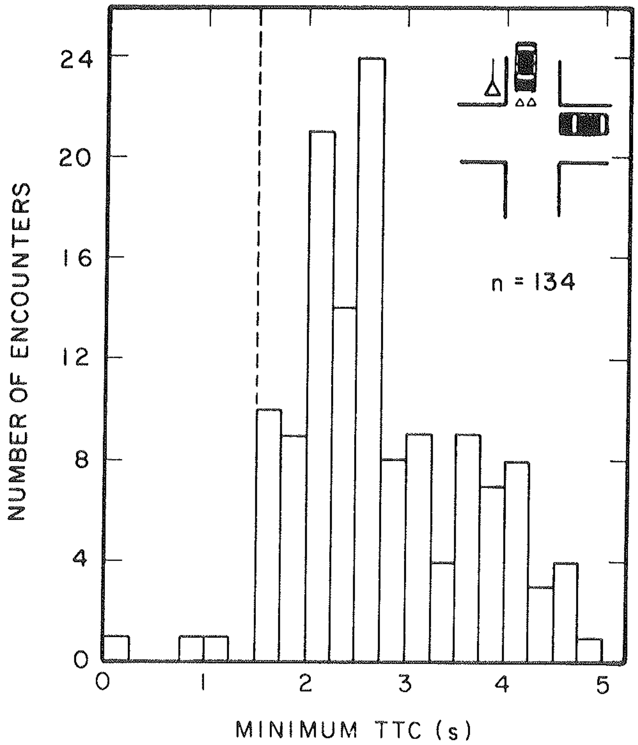


**FIGURE 5 Distribution of car-car encounters with a minimum *TTC* < 5 sec at a yield-sign intersection.**

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# Data Acquisition and Analysis System Based on a Lap-Top Computer

R. WADE ALLEN, JEFFREY R. HOGUE, THEODORE J. ROSENTHAL, AND ZAREH PARSEGHIAN

**Described in this paper is a data acquisition and analysis system based on a lap-top computer that is compatible with an IBM personal computer (PC). The hardware consists of a computer with a standard buss slot and analog-to-digital input/output card. The software permits acquiring up to eight analog signals of data, four of which can be plotted on the computer display screen as data are acquired. Data can be stored as standard ASCII files or in compact binary form to minimize file size and speed input/output. At the end of a data collection run, data can be scaled easily and plotted for review. Additional data reduction features permit data smoothing and crossplots between any pair of variables. The system replaces conventional tape recorders and strip chart recorders and stores data in a format convenient for further computer processing. The system can operate on 12-volt vehicle power and is compact and lightweight. The approach provides a convenient, lightweight, portable means to record and display vehicle motion and driver/rider response variables. Some data reduction can be accomplished online, and data can be easily processed and reviewed between runs to guide subsequent test conditions. Several popular lap-top computers with accessible PC busses are compatible with the approach. Some existing systems that could be used include the Datavue Snap 1+1 and the GRID Case Exp.**

Data acquisition is a routine requirement in field testing vehicle response and driver/rider behavior. Acquisition systems must be able to monitor, condition, record, and review data. In the past, these requirements were met with combinations of electronics, tape recorders, and strip chart recorders (*1*). Where minimum weight was required, telemetry was used to permit off-board recording (*2*).

With the advent of microprocessor and microcomputer technology, a variety of data acquisition systems have been developed. These systems include both special purpose microprocessor-based systems (*3*) and systems that incorporate standard or ruggedized commercial microcomputers (*4,5*). Commercial microcomputers permit access to low-cost technologies, including disk storage, analog-to-digital converters, and software. This approach has been taken with the system discussed below.

## SYSTEM DESCRIPTION

The data acquisition system is shown in Figure 1. It includes sensors, data conditioning electronics, and an acquisition and

Systems Technology, Inc., 13766 South Hawthorne Boulevard, Hawthorne, Calif. 90250-7083.

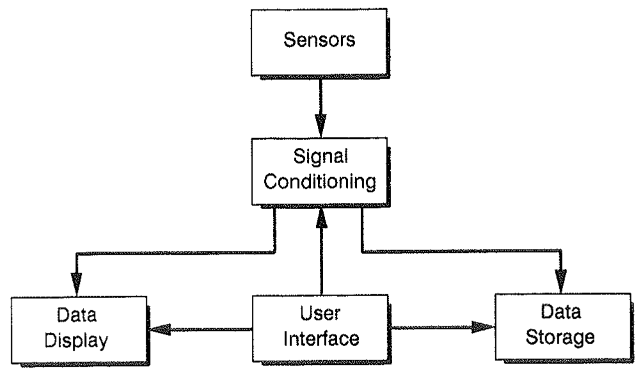



**FIGURE 1 Typical data acquisition system elements.**

storage computer. The sensors consist of a variety of conventional elements that are specified for a given application, including potentiometers, tachometers, accelerometers, rate gyros, pressure transducers, thermocouples, and fuel flow meters. Transducer outputs are conditioned with conventional analog electronics to supply filtered voltage signals for analog-to-digital (A/D) conversion.

Several A/D cards are available that will plug directly into microcomputer busses. The reason for configuring the current system was to obtain low-cost, lightweight hardware that could be configured conveniently with software. A variety of lightweight portable computers are available that are compatible with IBM personal computers (PCs) have accessible buss slots for installing expansion cards. A conventional portable computer, compatible with an IBM PC, was used for the prototype application, which involved automobile fuel economy tests. A Datavue Snap 1+1 lap-top computer was used for the second application, which involved lightweight, off-road vehicle testing. In both cases, a Metrobyte DAS-16 16-channel card with 12-bit resolution was used for A/D conversion.

The signal conditioning unit, shown in Figure 2, amplifies and filters sensor signals. Signals are amplified to give voltages within the range of 1 to 10 volts. The 12-bit resolution of the Metrobyte card then gives signal resolutions better than 0.5 percent. Filter break frequencies are set at least three times below the A/D sampling rate to avoid aliasing.

Software was developed to permit online sensor checkout and data monitoring, data recording and permanent storage, and data replay with limited data processing. The software allows the user to (a) specify signal scaling for up to eight recorded signals, (b) select up to four signals that can be monitored on the computer screen during data acquisition,

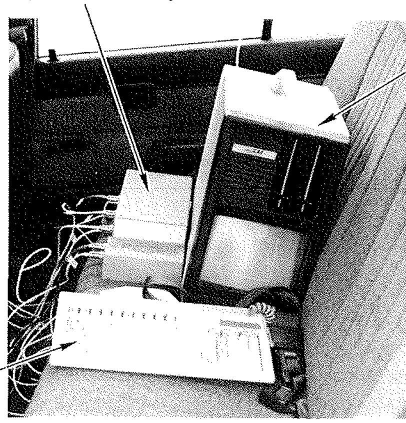




**FIGURE 2 Vehicle instrumentation for fuel economy tests.**

and (c) select all eight signals, four at a time, for post-run data review. The sampling rate can be specified up to 20 Hz for eight channels on a 4.7-MHz machine. This sampling capability is adequate for driver/rider behavior and most vehicle response variables. Higher sampling rates can be obtained with fewer signals and a faster processor clock rate (e.g., up to 500 Hz on a 16-MHz 80386 PC).

Run length varies up to the limit of memory, which is approximately 1.5 mins of data acquisition at 20 Hz for eight channels on a 640K byte machine. Run lengths can be increased by reducing the sampling rate and number of channels sampled. Extended memory can be added to expand data storage capability, which is provided by floppy disks. The current hardware configuration uses a machine with two 3½-in floppy disks that can store up to 720K bytes of data. Data input/output allows the user to write binary files as opposed to the standard ASCII file format. Binary files are 20 percent shorter than standard ASCII files and can be written and loaded five to seven times faster. The data files contain both raw data and scale factors, and file naming conventions can identify each file uniquely.

While the system is running, all data are stored in random access memory (RAM). When a test run is finished, data are written onto floppy disks. At the start of the next run, the program clears all of the previously used RAM memory. Although the data are written onto a disk after each run, the disk should be removed during test runs to prevent damage and possible loss of data.

## APPLICATIONS

The computer data acquisition system has been applied in two different field tests. The first application concerned fuel economy testing and vehicle performance was the only issue. This application had a low sample rate requirement (e.g., 1 to 2 samples per sec) and involved a prototype of the current data acquisition system. The second application involved off-road vehicle testing, both vehicle and driver behavior were of concern, and high sampling rates were important (e.g., 20 samples per sec).

### Fuel Economy

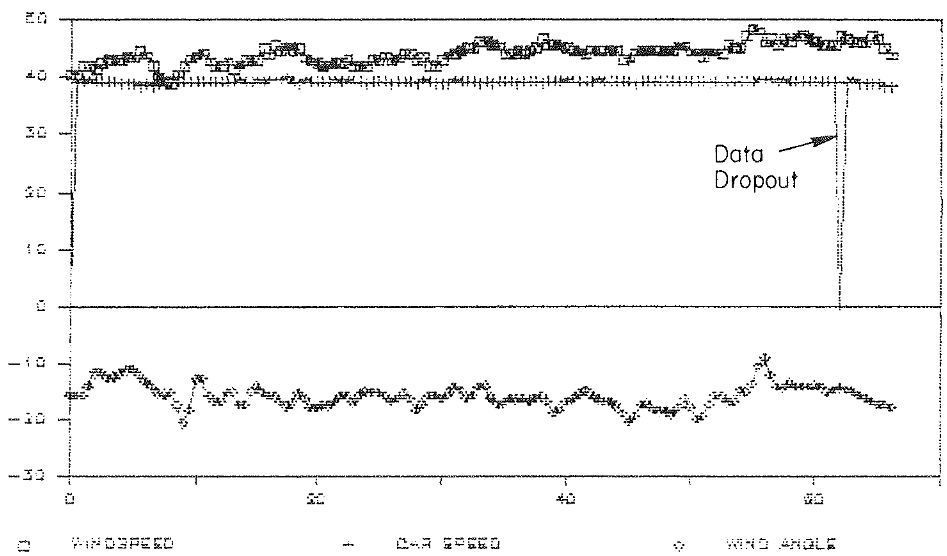
The fuel economy application required measurement of fuel flow, vehicle speed and distance traveled, and aerodynamic influences on the vehicle (wind velocity and direction). The vehicle was instrumented as illustrated in Figure 2. Vehicle forward motion was sensed with a magnetic pickup on the rear-mounted fifth wheel. Speed was derived from the time differential between pulses from the magnetic pickup. Distance traveled was obtained by counting pulses. Wind velocity and direction were sensed with a wind vane mounted on the front of the vehicle. Wind vane angle was sensed with a potentiometer and a tachometer measured propeller speed, which is proportional to wind speed. Fuel flow and total fuel consumed were sensed with a Maxmeter model 213 Flowmeter, which is designed to accurately measure small liquid volume displacements (i.e., produced 111 digital pulses per cc of fuel). Actual fuel usage must be measured in terms of weight, so fuel temperature was also measured with a thermocouple and an SAE formula was used to convert volume and temperature to mass flow.

The sensor signals were conditioned and filtered, consistent with a 2-Hz sample rate. In this prototype application, data were acquired using a conventional portable computer, compatible with an IBM PC, mounted on the seat of the test vehicle so it could be easily controlled by the driver. Fuel economy runs were conducted on various stretches of rural roads to determine the influence of pavement condition on fuel usage. The driver conducted several minute runs in both directions on given road segments to cancel out grade effects. Data files were stored in the field on standard floppy disks. Data review and analysis were conducted using a standard spread sheet program (i.e., Lotus 1-2-3). Data from a typical run are shown in Figure 3.

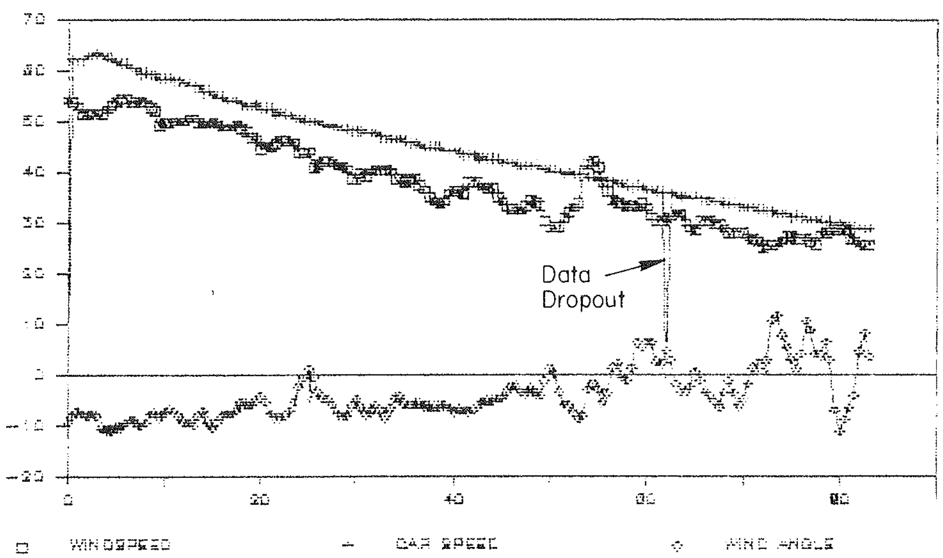
### Driver/Vehicle Response

The prototype approach was adapted to meet lightweight off-road vehicle applications. Instrumentation weight was the overriding concern in these applications, which suggested the use of a lap-top computer. The hardware arrangement on an instrumented vehicle is shown in Figure 4. The instrumented signals included vehicle speed, steering angle, yaw rate, and lateral and longitudinal acceleration. A string potentiometer was also configured to monitor rider lateral position, which could be shifted to affect vehicle motion.

Because of the extreme environmental conditions in the off-road application, a simple flexible plastic material enclo-




*a) Constant Speed Run*




*b) Coast Down Run*

**FIGURE 3 Example fuel economy test data.**

*a) Instrumented Vehicle*

*b) Rear Mounted Laptop Computer*

**FIGURE 4 Instrumented off-road vehicle.**

sure was prepared for the computer. The unit was mounted in a foam-lined fixture, as shown in Figure 4. Data acquisition, display, and storage could then be conducted reliably despite relatively hot, dusty, and vibratory conditions.

The objective of the off-road vehicle tests was to measure vehicle steering control characteristics and rider behavior during standard maneuvers and obstacle avoidance tests. Test data were routinely monitored on the lap-top computer screen to verify proper sensor operation and test conditions. Figure 5 shows the format for online data presentation during acquisition. Variable names assigned by the user are displayed on the left-hand side of the data plot, along with scale factors. Data are written to the screen by a cursor that moves from left to right and writes over data from the previous pass.

When a data collection run has been completed, the user can elect to replot the complete run and store the data as either a binary or ASCII file on floppy disk. Figure 6 shows the data plotting format for raw recorded data from a typical obstacle avoidance maneuver. The user can also request a full-scale plot of any one variable at a time, as illustrated in Figure 6. Further data processing can be carried out, including the data smoothing technique shown in Figure 7, and *X-Y* plots, as illustrated in Figure 8 for the two previously smoothed variables.

## CONCLUDING REMARKS

The current lap-top configuration of the instrumentation system is a very compact, economical, lightweight and rugged package for acquiring, displaying, and storing vehicle and driver/rider response data. The computer and A/D card weigh approximately 15 lbs. The primary data acquisition and display program is written in compiled QuickBASIC with math coprocessor (i.e., 8087) support. Application-specific processing needs can be easily added to the primary program or stored data files can be read into general purpose processing software such as spread sheet or relational data base programs.

A variety of lightweight lap-top computers with processor speeds up to 16 MHz are available and compatible with the

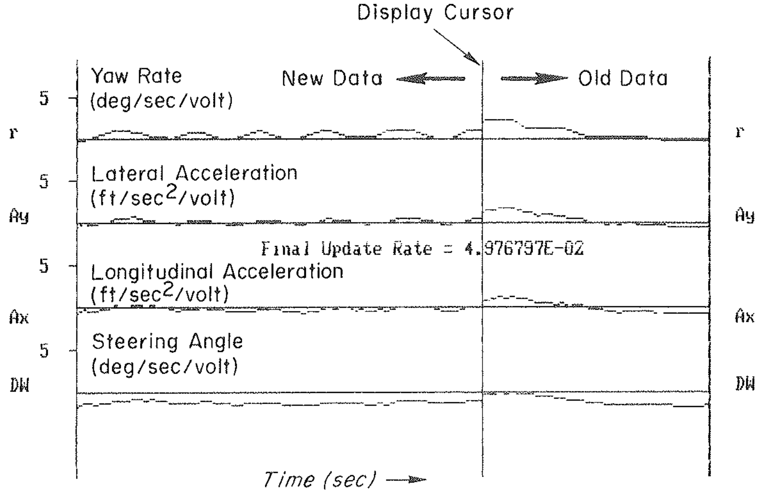
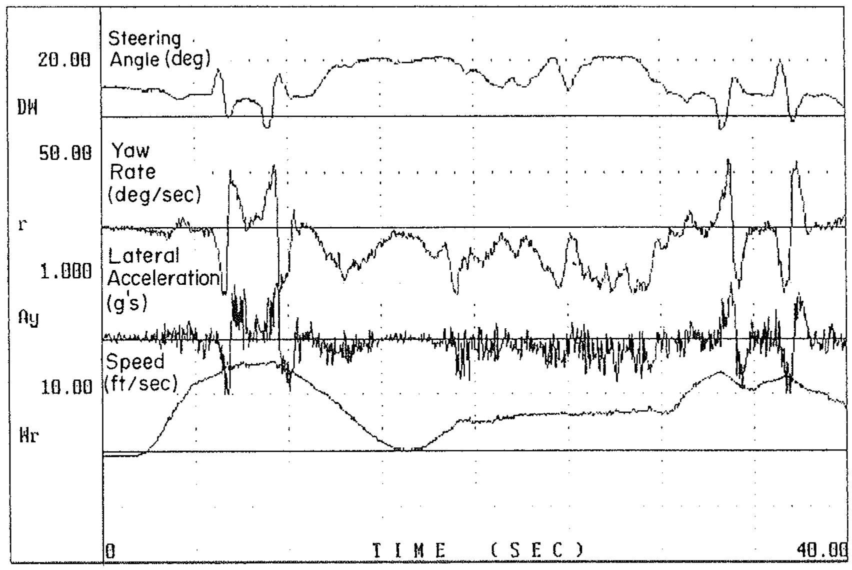
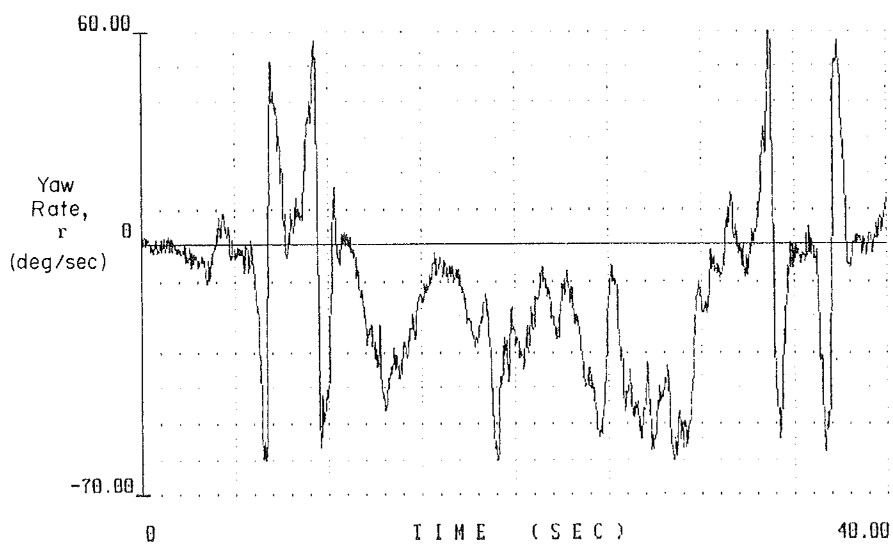



**FIGURE 5 Online data acquisition display.**

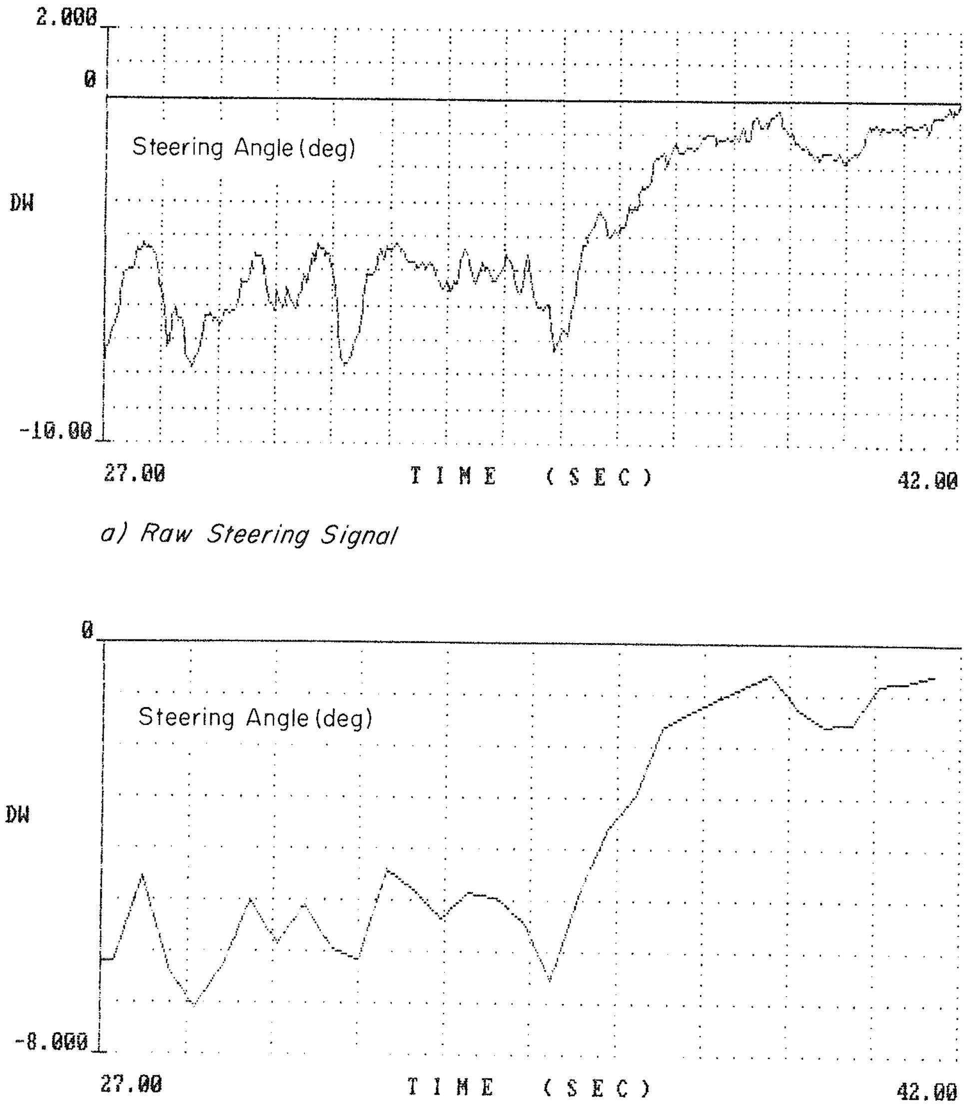



*a) Four Signal Format*



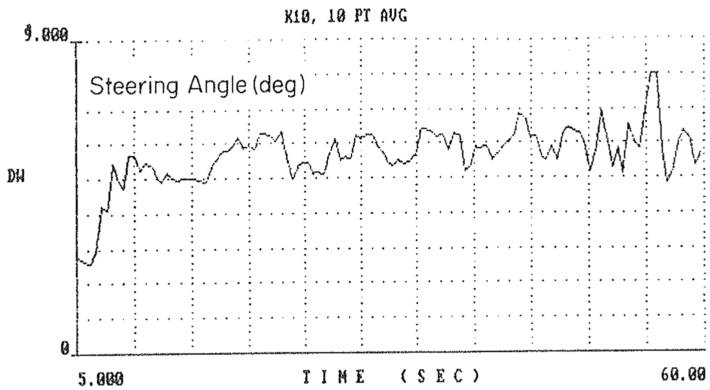

*b) Single Signal Expansion*

**FIGURE 6 Post-run raw data plotting formats.**

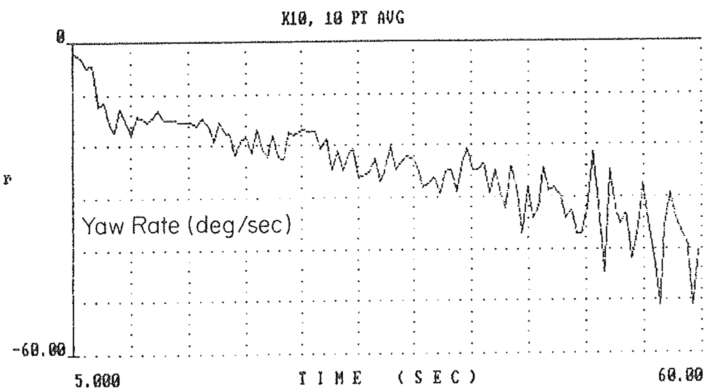



b) Smoothed Steering Signal, 10 Point Average

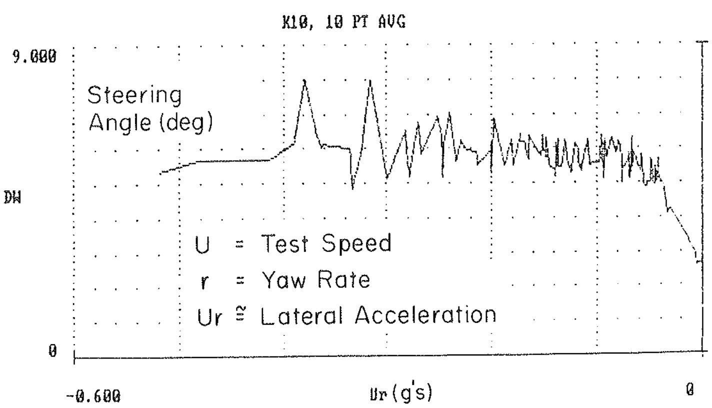
**FIGURE 7** **Signal smoothing using a zero phase lag window average.**




*a) Smoothed Steering Data*




*b) Smoothed Yaw Rate Data*




*c) Steering vs. Derived Lateral Acceleration*

**FIGURE 8** ***X-Y*** **data plot format for off-road vehicle test.**

approach discussed here. Benchmarks suggest that the new high-speed processors (i.e., 80286 and 80386) will permit significant performance improvements, including combinations of higher data rates (e.g., 500 Hz for each of eight signals), additional online processing, and more complex display formats. Higher-resolution screens will also permit richer data display formats.

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