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# RECORD

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*Safety and Human Performance*

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# Transportation Research Record 1376

## Contents

# Foreword

Research on several aspects of highway safety is presented in this Record. Heavy vehicles continue to pose a safety concern and prompt research. The first four papers cover structural geometric design, commercial driver, and accident countermeasure research related to heavy vehicles. Then driver licensing manuals are examined from the perspective of traffic engineering. Variable message signs are evaluated from a driver's perceptual point of view, high-speed isolated signalized intersections are studied, and motorists' understanding of left-turn signals and auxiliary signs is assessed. The total station approach to incident management is discussed. The effect of intersection congestion on accident rates and ways to estimate accident rates are studied, and a health department that identified dangerous highway locations is reported. Finally, development of a quantitative measure of traffic conflicts is described.

# Structural Analyses of Two Typical Medium-Duty Transit Buses

RALPH A. DUSSEAU, SNEHAMAY KHASNABIS, AND SAMI M. ZAHER

Finite-element computer models were developed for two medium-duty transit buses: a 21-ft bus with 11 seats (22 passengers) and a 25-ft bus with 13 seats (26 passengers). Two models of each bus were derived: one with passenger seats fastened to the bus floor only and one with seats attached to both the bus sidewalls and the floor. The models were each analyzed under three cases of bus deceleration: with seat belts installed and used on all passenger seats, with seat belts installed on all seats but used by approximately half of the bus passengers, and with seat belts installed and used on the front seats only. Each load case was analyzed using seven bus floor angles from 0 to 30 degrees. The following conclusions were reached with respect to the structural responses of a typical medium-duty transit bus to bus deceleration: (a) maximum member stresses should generally be lower with full versus staggered seat belt use or versus front seat belt use only; (b) maximum member stresses should generally be higher with seats attached to both the sidewalls and the floor versus seats fastened to the floor only; (c) maximum member stresses could be relatively high in the seat anchorage members for wall- and floor-mounted seats and in the perimeter frame members for floor-mounted only seats; and (d) differences should be relatively small between the maximum member stresses for shorter versus longer medium-duty transit buses.

A study to assess the structural responses of medium-duty transit buses subjected to various levels of bus deceleration is currently under way at the Department of Civil Engineering, Wayne State University. The principal objective of this investigation is to perform parametric analyses with various combinations of seat belt use and seat mounting in order to measure any differential stresses that might be generated in the structural members of the buses under passenger inertial forces caused by bus deceleration.

A comprehensive literature review conducted as a part of the project showed very little research to assess the behavior of the structural components of a bus frame under bus deceleration. Reports dealing with front-end crash tests of school and transit buses have concentrated on "visible" damage, including passenger seat detachment from the floors (*1–4*), slippage of the frame-to-chassis connections (*1,5,6*), and buckling of the floor (*1,2,4*). The crash responses of the remaining structural components of the buses tested were not reported, however.

One previously reported use of finite-element computer modeling in the analysis of transit buses was a series of models developed by DAF Trucks, Eindhoven, the Netherlands (*7*). The goal was to measure the effects of bending stiffness and torsional stiffness on the dynamic responses and hence the ride comfort of passengers. No analyses under bus deceleration were performed, however.

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The work presented here is a continuation of the research conducted by Dusseau et al. (*8,9*). That effort involved finite-element analysis of the structure of a 25-ft transit bus that included the frame, floor, and chassis. Assumptions were made about the loading conditions under bus deceleration. Parametric results for floor angles from 0 to 30 degrees at maximum deceleration were derived for floor-mounted seats using two loading patterns: with seat belts installed and used on all passenger seats and with seat belts installed and used on the front seats only. It was found that the structural members in the frame could experience moderate to substantial decreases in maximum stress if seat belts were installed and used on all seats, whereas the maximum stresses in the chassis members could be slightly higher to moderately higher if seat belts were installed and used on all seats.

In the present study, finite-element computer models were developed for two medium-duty transit buses: a 21-ft bus with 11 seats and a capacity of 22 passengers and a 25-ft bus with 13 seats and a capacity of 26 passengers. Two finite-element models were derived for each transit bus studied: one with passenger seats fastened to the floor only (model with floor-mounted seats) and one with seats attached to both the sidewalls and the floor (model with wall-mounted seats). The four bus models were each analyzed under three cases of bus deceleration: with seat belts installed and used on all seats (full seat belt use), with seat belts installed on all seats and used by about half of the passengers (staggered seat belt use), and with seat belts installed and used on the front seats only (front seat belt use only). Results using seven angles of tilt from 0 to 30 degrees for the bus floor at maximum deceleration were derived for each load case.

The major additions in the present study compared with the previous investigation are (a) the analysis of the 21-ft bus; (b) the inclusion of the sidewalls, backwall, and roof for each model; (c) the analysis of models with wall-mounted seats; and (d) the load case with staggered seat belt use.

## MODELS AND ASSUMPTIONS

The 21-ft bus is a shorter version of the 25-ft bus with two fewer seats and about 4 ft less chassis, frame, floor, and body. The same chassis and axle spacing are used for both buses, however. All of the steel members in the frame, chassis, body, and seats are cold-formed steel sections with minimum yield stresses of 30,000 psi. The floor is composed of exterior grade plywood with an estimated yield stress of 2,500 psi. The floor has steel plate reinforcing along the lines where the interior legs of the seats are bolted to the floor and along the plywood seam that follows the centerline of the floor. Steel plate is also used in the tops of the rear wheel wells.

The floor is supported by lateral frame members fabricated from channel sections; these run between the sidewalls and support the body, floor, and frame. Angle sections are used for the skirting and other frame members around the perimeter of the floor. The lateral frame members are welded to longitudinal chassis caps fabricated from channel sections and are attached to the chassis with U-bolt connections. The chassis is composed of two longitudinal members fabricated from channel sections and are connected at intervals by lateral chassis members also fabricated from channel sections. The body is fabricated from square tubes and channel sections, and the seats are fabricated from square tubes and steel plates. The floor-mounted seats have two inverted T-legs with the interior legs fastened to the floor and the exterior legs fastened to the perimeter of the frame. The wall-mounted seats are similar to the floor-mounted seats but with the exterior legs deleted and the exterior edges of the seats fastened to seat anchorage members that run the length of the bus body.

The simplifications and assumptions made in developing the bus models were as follows:

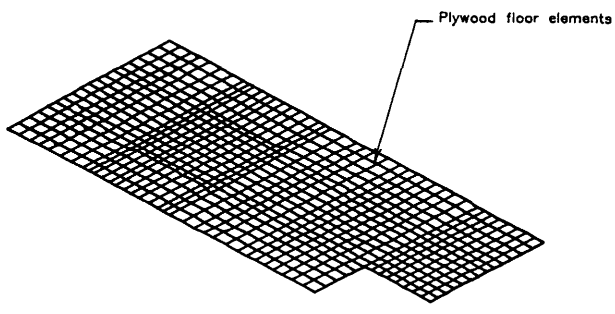


**FIGURE 1 Plywood floor elements for 21-ft bus.**

1. Because the goal of the research was to assess the relative effects of seat mounting and seat belt use on the dynamic responses of the transit buses modeled, two key simplifications were made in modeling the buses: (a) only the inertial forces due to the passengers were considered in the analyses, and (b) the front portion of the body, the stairs, the battery tray, and other minor structural members that contribute little to the stiffness and strength of the bus structure were excluded from the models.
2. The plywood floor was modeled using plate finite elements as depicted in Figure 1 for the 21-ft bus. Because the plywood floor was modeled without seams, the steel plate reinforcing along the centerline of the floor was not included

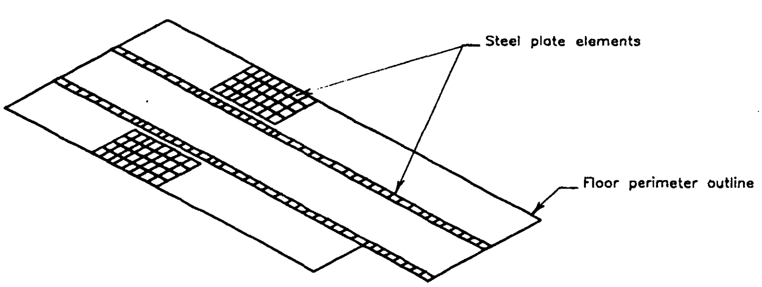


**FIGURE 2 Steel plate elements for 21-ft bus.**

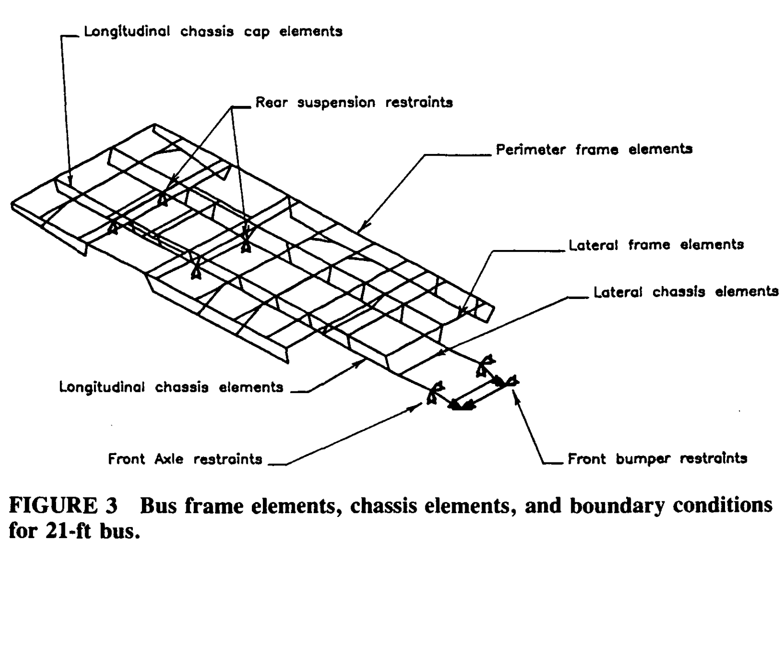


**FIGURE 3 Bus frame elements, chassis elements, and boundary conditions for 21-ft bus.**

in the model. The steel plate reinforcing along the bolt line of the interior seat legs and in the rear wheel wells was modeled using plate elements as shown in Figure 2 for the 21-ft bus.

3. The lateral frame members, perimeter frame members, and longitudinal chassis caps were all modeled using beam finite elements as illustrated in Figure 3 for the 21-ft bus. For simplicity, the centroids of these beam elements were all placed in the same horizontal plane as the plywood floor. The longitudinal chassis members, lateral chassis members, and skirting members were also modeled using beam elements as depicted in Figure 3. Also shown in Figure 3 are semirigid (high-stiffness) elements that were used to connect the centroids of the longitudinal chassis members with the lateral frame members at the points at which the lateral frame members are welded to the longitudinal chassis caps.

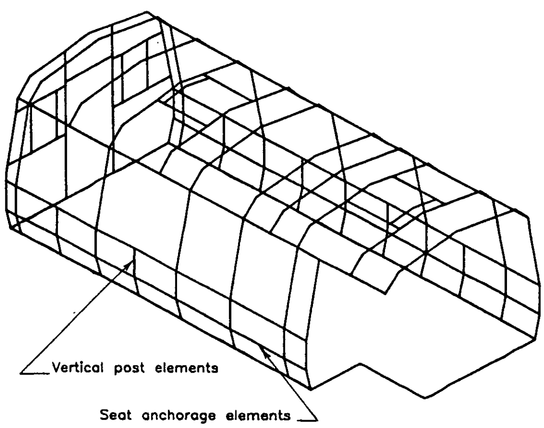


FIGURE 4 Bus body elements for 21-ft bus.

4. The sidewalls, backwall, and roof members were modeled using beam elements as depicted in Figure 4 for the 21-ft bus model.

5. The front axle is assumed to bottom out under bus deceleration. Therefore (as shown in Figure 3), the buses were modeled with vertical and lateral restraints at the points at which rubber stops are attached to the longitudinal chassis members to prevent damage due to bottoming out of the front axle. Longitudinal and lateral restraints were used at the front of the longitudinal chassis members where the front bumper is attached, and vertical restraints were used at the points at which the rear leaf springs are attached to the longitudinal chassis members.

6. Each floor-mounted and wall-mounted seat was represented by five semirigid members that were arranged like a swingset with one horizontal element connecting the nodal points representing the centers of gravity (CGs) of the two

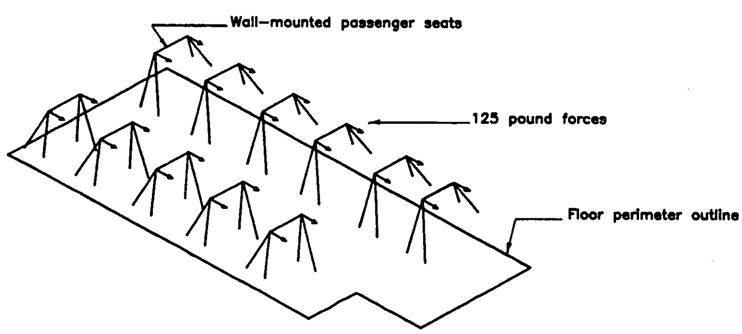


FIGURE 5 Passenger seats and load application for 21-ft bus with wall-mounted seats and full seat belt use.

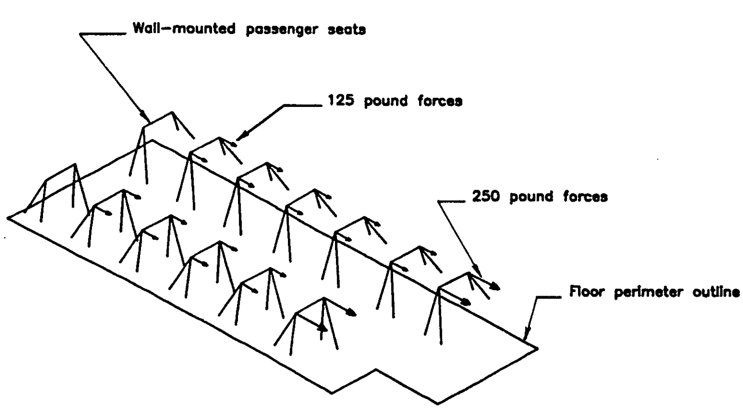


FIGURE 6 Passenger seats and load application for 25-ft bus with wall-mounted seats and front seat belt use only.

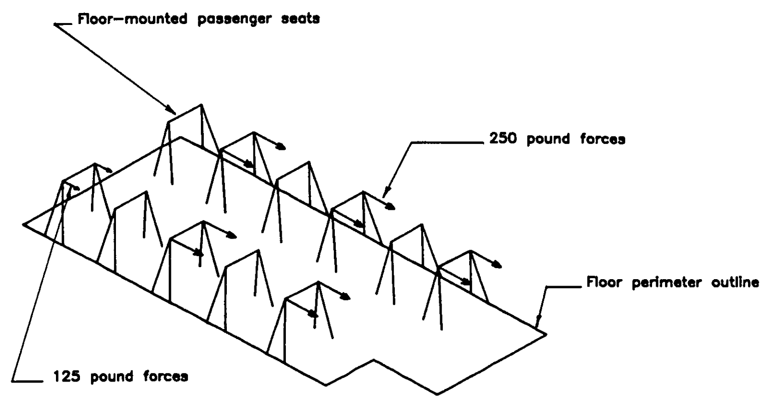


**FIGURE 7 Passenger seats and load application for 21-ft bus with floor-mounted seats and staggered seat belt use.**

passengers in the seat and two diagonal elements connecting each of these CG points to the floor or sidewalls at or near the points at which the actual seats are attached. Figures 5 and 6 depict the 21- and 25-ft buses, respectively, with wall-mounted seats, and Figure 7 shows the 21-ft bus with floor-mounted seats.

7. The finite-element program used for the investigation was the ANSYS program developed by Swanson Analysis Systems, Inc., Houston, Pennsylvania.

## LOAD CASES

An average weight of 125 lbs was assumed for each bus passenger on the basis of a mix of adults and children. Thus, to simulate the loads generated by passenger inertia under a 1 *g* bus deceleration, a force of 125 lb/bus passenger was used. These forces were applied using seven angles of tilt from 0 to 30 degrees for the bus floor at maximum deceleration. These angles of tilt were simulated by "tilting" the forces as opposed to tilting the models.

The loading pattern used to represent bus deceleration with full seat belt use consisted of two 125-lb forces applied to each passenger seat (as shown in Figure 5 for the 21-ft bus with wall-mounted seats). For load cases with unbelted passengers, a 125-lb force was applied to the seat in front of each unbelted passenger. Thus, for bus deceleration with front seat belt use only (as depicted in Figure 6 for the 25-ft bus with wall-mounted seats) no forces were applied to the rear seats, two 125-lb forces were applied to each intermediate seat, and two 250-lb forces were applied to each front seat. For bus deceleration with staggered seat belt use, a checkerboard loading pattern (as depicted in Figure 7 for the 21-foot bus with floor-mounted seats) was used.

## ANALYSIS RESULTS

Table 1 gives the 12 load cases analyzed; Table 2 gives the maximum element stresses of bus deceleration and the corresponding floor angles; and Table 3 gives the lateral and longitudinal locations of the maximum element stresses. The longitudinal locations in Table 3 are measured along the centerline of the bus beginning at the back and are normalized with respect to the bus length. Thus, the longitudinal location 0.00 refers to the point at which the rear bumper is attached, and the location 1.00 refers to the point at which the front bumper is attached. The lateral locations in Table 3 are measured from the centerline of the bus and are normalized with respect to the half-width of the floor. Thus, the lateral location −1.00 refers to the left edge of the floor and the lateral location +1.00 refers to the right edge.

### Analysis Limitations

The analysis results in Table 2 have certain limitations based on the modeling assumptions used in the analyses. These limitations are centered on the maximum levels of bus deceleration for which the analysis results are valid. The assumptions that control these limiting values of bus deceleration involve the applied load, the linear elastic analysis procedure, and the boundary conditions.

As discussed, because the finite-element analyses were primarily aimed at determining the effects on maximum member stresses caused by seat belt use and seat mounting, the inertia of the bus members and the bus components and the gravitational forces generated by the bus and the bus passengers were not considered in the analyses. These forces could play a role in determining the level of bus deceleration at which member yielding first occurs and hence the level of deceleration at which the linear elastic analysis results are no longer valid, but the authors believe that the effects of these forces will not be a major factor in this determination. This is because nearly all of the members that yield first are those that are directly connected to the bus seats and hence are most affected by passenger inertia. On the basis of the effects of passenger inertia only, the levels of bus deceleration at which member yielding first occurs and hence the maximum level of deceleration for which the linear elastic analyses are valid are given in Table 1.

The boundary conditions for the front bumper and front axle locations appear to be valid under all levels of bus deceleration, but the vertical restraints at the rear spring locations may not be. As shown by crash test videos of school buses, large front-end collisions can cause the rear wheels to

TABLE 1 Bus Load Cases and Limiting Values of Bus Deceleration

| LOAD CASE | BUS VERSION AND SEAT TYPE | SEAT BELT USAGE | BUS DECELERATION LIMITS, g | | |
|---|---|---|---|---|---|
| | | | ELASTIC FINITE-ELEMENT MODELS | BOUNDARY CONDITIONS (BUS FLOOR ANGLE) | |
| | | | | 0 DEGREES | 30 DEGREES |
| 21F1 | 21-Foot Bus with Floor-Mounted Seats | Full Seat Belt Usage | 10.6 | 27.2 | 6.9 |
| 21F2 | 21-Foot Bus with Floor-Mounted Seats | Staggered Seat Belt Usage | 6.0 | 24.0 | 7.0 |
| 21F3 | 21-Foot Bus with Floor-Mounted Seats | Front Seat Belt Usage Only | 6.6 | 21.9 | 6.9 |
| 21W1 | 21-Foot Bus with Wall-Mounted Seats | Full Seat Belt Usage | 5.7 | 28.1 | 7.0 |
| 21W2 | 21-Foot Bus with Wall-Mounted Seats | Staggered Seat Belt Usage | 3.1 | 26.3 | 7.1 |
| 21W3 | 21-Foot Bus with Wall-Mounted Seats | Front Seat Belt Usage Only | 4.7 | 23.8 | 7.1 |
| 25F1 | 25-Foot Bus with Floor-Mounted Seats | Full Seat Belt Usage | 12.8 | 27.2 | 5.9 |
| 25F2 | 25-Foot Bus with Floor-Mounted Seats | Staggered Seat Belt Usage | 6.3 | 23.8 | 5.9 |
| 25F3 | 25-Foot Bus with Floor-Mounted Seats | Front Seat Belt Usage Only | 6.5 | 21.8 | 5.9 |
| 25W1 | 25-Foot Bus with Wall-Mounted Seats | Full Seat Belt Usage | 8.5 | 26.9 | 5.9 |
| 25W2 | 25-Foot Bus with Wall-Mounted Seats | Staggered Seat Belt Usage | 4.4 | 24.7 | 5.9 |
| 25W3 | 25-Foot Bus with Wall-Mounted Seats | Front Seat Belt Usage Only | 4.4 | 22.7 | 5.9 |

TABLE 2 Maximum Element Stresses and Corresponding Bus Floor Angles Versus Bus Load Cases

| ELEMENT DESCRIPTIONS | MAXIMUM ELEMENT STRESSES PER G (ksi/g) / CORRESPONDING BUS FLOOR ANGLES (degrees) | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | LOAD CASE 21F1 | LOAD CASE 21F2 | LOAD CASE 21F3 | LOAD CASE 21W1 | LOAD CASE 21W2 | LOAD CASE 21W3 | LOAD CASE 25F1 | LOAD CASE 25F2 | LOAD CASE 25F3 | LOAD CASE 25W1 | LOAD CASE 25W2 | LOAD CASE 25W3 |
| Primary Structural Members | | | | | | | | | | | | |
| Plywood Floor Elements | 0.069<br>0 | 0.076<br>0 | 0.081<br>0 | 0.069<br>0 | 0.115<br>0 | 0.117<br>0 | 0.069<br>0 | 0.075<br>0 | 0.083<br>0 | 0.071<br>0 | 0.140<br>0 | 0.113<br>0 |
| Lateral Frame Elements | 1.43<br>0 | 2.35<br>0 | 2.26<br>0 | 1.36<br>10 | 2.28<br>0 | 1.60<br>0 | 1.47<br>0 | 2.12<br>0 | 2.16<br>0 | 1.41<br>0 | 1.48<br>0 | 1.61<br>0 |
| Longitudinal Chassis Elements | 1.09<br>0 | 1.30<br>0 | 1.35<br>0 | 1.44<br>30 | 1.59<br>30 | 1.91<br>30 | 1.71<br>25 | 1.66<br>20 | 1.26<br>0 | 1.56<br>25 | 1.29<br>15 | 1.66<br>30 |
| Secondary Structural Members | | | | | | | | | | | | |
| Body Elements | 1.96<br>10 | 2.55<br>0 | 2.72<br>0 | 5.29<br>30 | 9.65<br>30 | 6.44<br>0 | 2.35<br>30 | 3.71<br>15 | 1.99<br>15 | 3.52<br>30 | 6.89<br>5 | 6.87<br>30 |
| Steel Plate Elements | 0.72<br>0 | 0.80<br>0 | 0.85<br>0 | 0.86<br>0 | 1.50<br>0 | 1.52<br>0 | 0.73<br>30 | 0.79<br>0 | 0.87<br>0 | 0.86<br>0 | 1.58<br>0 | 1.55<br>0 |
| Perimeter Frame Elements | 2.82<br>0 | 4.99<br>0 | 4.55<br>15 | 1.16<br>0 | 1.59<br>0 | 1.24<br>30 | 2.31<br>0 | 4.80<br>0 | 4.60<br>15 | 1.50<br>30 | 1.52<br>30 | 1.04<br>30 |
| Longitudinal Chassis Cap Elements | 1.11<br>30 | 1.13<br>0 | 0.77<br>0 | 1.15<br>0 | 1.61<br>0 | 0.98<br>30 | 1.13<br>30 | 0.87<br>30 | 0.85<br>30 | 1.23<br>30 | 1.20<br>30 | 1.08<br>30 |
| Lateral Chassis Elements | 0.66<br>30 | 0.87<br>30 | 0.77<br>30 | 0.43<br>30 | 0.40<br>30 | 0.49<br>30 | 0.38<br>30 | 0.58<br>30 | 0.51<br>30 | 0.34<br>30 | 0.52<br>30 | 0.34<br>30 |

**TABLE 3 Longitudinal and Lateral Locations Corresponding to Maximum Element Stresses**

| ELEMENT DESCRIPTIONS | LONGITUDINAL LOCATIONS / LATERAL LOCATIONS | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | LOAD CASE 21F1 | LOAD CASE 21F2 | LOAD CASE 21F3 | LOAD CASE 21W1 | LOAD CASE 21W2 | LOAD CASE 21W3 | LOAD CASE 25F1 | LOAD CASE 25F2 | LOAD CASE 25F3 | LOAD CASE 25W1 | LOAD CASE 25W2 | LOAD CASE 25W3 |
| Primary Structural Members | | | | | | | | | | | | |
| Plywood Floor Elements | 0.82<br>-0.40 | 0.82<br>-0.40 | 0.82<br>-0.40 | 0.29<br>-0.46 | 0.69<br>-0.50 | 0.69<br>-0.50 | 0.84<br>-0.40 | 0.84<br>-0.40 | 0.84<br>-0.40 | 0.18<br>0.50 | 0.18<br>0.50 | 0.69<br>-0.31 |
| Lateral Frame Elements | 0.82<br>-0.45 | 0.82<br>-0.45 | 0.82<br>-0.45 | 0.33<br>-0.43 | 0.45<br>-0.45 | 0.55<br>-0.45 | 0.84<br>-0.35 | 0.84<br>-0.45 | 0.84<br>-0.45 | 0.84<br>-0.35 | 0.84<br>-0.35 | 0.84<br>-0.35 |
| Longitudinal Chassis Elements | 0.84<br>-0.35 | 0.84<br>-0.35 | 0.84<br>-0.35 | 0.53<br>-0.35 | 0.53<br>-0.35 | 0.53<br>-0.35 | 0.29<br>0.35 | 0.23<br>0.35 | 0.96<br>0.33 | 0.29<br>0.35 | 0.29<br>0.35 | 0.58<br>-0.35 |
| Secondary Structural Members | | | | | | | | | | | | |
| Body Elements | 0.00<br>1.03 | 0.65<br>-1.03 | 0.65<br>-1.03 | 0.40<br>-1.06 | 0.40<br>-1.06 | 0.72<br>-1.06 | 0.18<br>-1.03 | 0.18<br>1.03 | 0.68<br>-1.00 | 0.68<br>-1.06 | 0.12<br>1.06 | 0.68<br>-1.06 |
| Steel Plate Elements | 0.82<br>-0.40 | 0.82<br>-0.40 | 0.82<br>-0.40 | 0.59<br>-0.40 | 0.59<br>-0.40 | 0.65<br>-0.40 | 0.84<br>-0.40 | 0.84<br>-0.40 | 0.84<br>-0.40 | 0.15<br>-0.40 | 0.15<br>0.40 | 0.69<br>-0.40 |
| Perimeter Frame Elements | 0.10<br>-1.00 | 0.47<br>-1.00 | 0.64<br>-1.00 | 0.20<br>-1.00 | 0.49<br>-1.00 | 0.64<br>-1.00 | 0.68<br>-1.00 | 0.19<br>1.00 | 0.68<br>-1.00 | 0.28<br>-1.00 | 0.28<br>1.00 | 0.66<br>-1.00 |
| Longitudinal Chassis Cap Elements | 0.03<br>-0.40 | 0.50<br>-0.40 | 0.18<br>0.40 | 0.18<br>-0.40 | 0.50<br>-0.40 | 0.34<br>-0.40 | 0.26<br>0.40 | 0.26<br>-0.40 | 0.40<br>-0.40 | 0.40<br>-0.40 | 0.40<br>0.40 | 0.40<br>-0.40 |
| Lateral Chassis Elements | 0.34<br>0.00 | 0.34<br>0.00 | 0.34<br>0.00 | 0.34<br>0.00 | 0.51<br>0.00 | 0.34<br>0.00 | 0.42<br>0.00 | 0.24<br>0.00 | 0.57<br>0.00 | 0.24<br>0.00 | 0.24<br>0.00 | 1.00<br>0.00 |

lift off the ground. Thus, at high levels of bus deceleration, the vertical restraints at the rear spring locations may no longer be valid for the models presented here. During the course of the analyses, the reactions at these locations were carefully monitored and recorded. Assuming that the rear of the bus will be held down by a gravitational force of 11,000 lb, which is the maximum capacity of the rear axle, the maximum bus decelerations required before the reactions at the rear spring locations exceed this 11,000-lb limit are given in Table 1 for bus floor angles of 0 and 30 degrees. Although the bus inertia could play a role in determining the level of bus deceleration beyond which the assumed boundary conditions are no longer valid, the authors believe that because much of the mass of the bus chassis is at or below the level of attachment of the rear springs, the bus inertia will not be a major factor in this determination.

## Primary Structural Members

The floor-frame-chassis system is the primary structural system that provides strength and stiffness for the transit buses modeled. The plywood floor members, lateral frame members, and longitudinal chassis members were thus classified as primary structural members on the basis of their relative size, location, and importance as members of the floor-frame-chassis system.

### *Plywood Floor Elements*

For the plywood floor elements, the most severe case was the 25-ft bus with wall-mounted seats and staggered seat belt use (25W2) at a floor angle of 0 degrees. The maximum stress of 0.140 ksi/*g* for this case was 97 percent higher than full seat belt use (25W1), 24 percent higher than front seat belt use only (25W3), 87 percent higher than floor-mounted seats (25F2), and 22 percent higher than the 21-ft bus (21W2). The maximum stresses for Case 25W2 and two other cases occurred near the rear wheel wells. The skirting members and other perimeter frame members are discontinuous at the rear wheel wells. The maximum stresses for six cases were near the left front passenger seat. The loads acting on the front seats are doubled for cases with staggered seat belt use and with front seat belt use only. The maximum stresses for the remaining three cases occurred between the left rear wheel well and the left front seat.

### *Lateral Frame Elements*

The most severe case for the lateral frame elements was the 21-ft bus with floor-mounted seats and staggered seat belt use (21F2) at a floor angle of 0 degrees. For this case, the maximum stress of 2.35 ksi/*g* was 64 percent larger than full seat belt use (21F1), 4 percent larger than front seat belt use only

(21F3), 3 percent larger than wall-mounted seats (21W2), and 11 percent larger than the 25-ft bus (25F2). The maximum stresses occurred near the left front seat for Case 21F2 and eight others, and between the left rear wheel well and the left front seat for three cases.

### *Longitudinal Chassis Elements*

For the longitudinal chassis elements, the worst case was the 21-ft bus with wall-mounted seats and front seat belt use only (21W3) at a floor angle of 30 degrees. The maximum stress of 1.91 ksi/*g* for this case was 33 percent higher than full seat belt use (21W1), 20 percent higher than staggered seat belt use (21W2), 41 percent higher than floor-mounted seats (21F3), and 15 percent higher than the 25-ft bus (25W3). The maximum stresses occurred between the left rear wheel well and the left front seat for Case 21W3 and three other cases, near the right rear wheel well for four cases, and near the front seats for four cases.

## Secondary Structural Members

Because they contribute less to the strength and stiffness of the buses that were modeled and hence are of less overall importance to the structure of these buses, the following were classified as secondary structural members: the body members, steel plate members, perimeter frame members, longitudinal chassis caps, and lateral chassis members.

### *Body Elements*

The worst case for the body elements was the 21-ft bus with wall-mounted seats and staggered seat belt use (21W2) at a floor angle of 30 degrees. For this case, the maximum stress of 9.65 ksi/*g* was 83 percent larger than full seat belt use (21W1), 50 percent larger than front seat belt use only (21W3), 278 percent larger than floor-mounted seats (21F2), and 40 percent larger than the 25-ft bus (25W2). For all six cases with wall-mounted seats, the maximum stresses occurred in the seat anchorage members. For the cases with floor-mounted seats, five cases had maximum stresses in the vertical posts below the windows and one case had maximum stress along the left edge of the frame.

### *Steel Plate Elements*

For the steel plate elements, the most severe case was the 25-ft bus with wall-mounted seats and staggered seat belt use (25W2) at a floor angle of 0 degrees. The maximum stress of 1.58 ksi/*g* for this case was 84 percent higher than full seat belt use (25W1), 2 percent higher than front seat belt use only (25W3), 100 percent higher than floor-mounted seats (25F2), and 5 percent higher than the 21-ft bus (21W2). The maximum stresses occurred near the rear wheel wells for Case 25W2 and one other case, near the left front seat for six cases, and between the left rear wheel well and the left front seat for four cases.

### *Perimeter Frame Elements*

The most severe case for the perimeter frame elements was the 21-ft bus with floor-mounted seats and staggered seat belt use (21F2) at a floor angle of 0 degrees. For this case, the maximum stress of 4.99 ksi/*g* was 77 percent larger than full seat belt use (21F1), 10 percent larger than front seat belt use only (21F3), 214 percent larger than wall-mounted seats (21W2), and 4 percent larger than the 25-ft bus (25F2). The maximum stress occurred between the left rear wheel well and the left front seat for Case 21F2 and six other cases, near the rear wheel wells for four cases, and near the left rear seat for one case.

### *Longitudinal Chassis Cap Elements*

For the longitudinal chassis cap elements, the worst case was the 21-ft bus with wall-mounted seats and staggered seat belt use (21W2) at a floor angle of 0 degrees. The maximum stress of 1.61 ksi/*g* for this case was 40 percent higher than full seat belt use (21W1), 64 percent higher than front seat belt use only (21W3), 42 percent higher than floor-mounted seats (21F2), and 34 percent higher than the 25-ft bus (25W2). The maximum stresses occurred between the rear wheel wells and the front seats for Case 25W2 and six others, near the rear wheel wells for four cases, and near the left rear seat for one case.

### *Lateral Chassis Elements*

The most severe case for the lateral chassis elements was the 21-ft bus with floor-mounted seats and staggered seat belt use (21F2) at a floor angle of 30 degrees. For this case, the maximum stress of 0.87 ksi/*g* was 32 percent larger than full seat belt use (21F1), 13 percent larger than front seat belt use only (21F3), 118 percent larger than wall-mounted seats (21W2), and 50 percent larger than the 25-ft bus (25F2). The maximum stresses occurred between the rear wheel wells and the front seats for Case 21F2 and seven others, at the rear wheel wells for three cases, and at the front of the bus for one case.

## SUMMARY AND CONCLUSIONS

Four finite-element computer models were developed for the structure of two typical medium-duty transit buses using floor- and wall-mounted seats. Assumptions were made regarding the loading conditions in the event of bus deceleration. Parametric results for floor angles of 0 to 30 degrees at maximum deceleration were derived for loading patterns with full seat belt use, staggered seat belt use, and front seat belt use only.

The following conclusions pertain to the bus responses with staggered and front seat belt use only versus full seat belt use:

1. For the plywood floor elements and the lateral frame elements, the load cases with staggered seat belt use and front

seat belt use only had slightly higher (+5 percent) to substantially higher (+97 percent) maximum stresses than full seat belt use.

2. The longitudinal chassis elements in the 21-ft bus had a slightly higher (+10 percent) to moderately higher (+33 percent) maximum stresses with staggered seat belt use and front seat belt use only versus full seat belt use.

3. For the longitudinal chassis elements in the 25-ft bus models, the load cases with staggered seat belt use and front seat belt use only had moderately lower (−26 percent) to slightly higher (+6 percent) maximum stresses compared with full seat belt usage.

4. The secondary structural members had moderately lower (−31 percent) to substantially higher (+108 percent) maximum stresses with staggered seat belt use and front seat belt use only versus full seat belt use.

The following conclusions pertain to the bus responses with wall- versus floor-mounted seats:

1. The maximum plywood floor element stresses per *g* were slightly higher (+1 percent) to substantially higher (+87 percent) with wall-mounted seats than floor-mounted seats.

2. The lateral frame elements had slightly lower (−3 percent) to moderately lower (−30 percent) maximum stresses with wall-mounted seats than with floor-mounted seats.

3. In the 21-ft bus, the longitudinal chassis elements had maximum stresses that were moderately higher (+22 percent) to substantially higher (+41 percent) with wall-mounted seats than floor-mounted seats.

4. The longitudinal chassis elements in the 25-ft bus had maximum stresses that were moderately lower (−22 percent) to moderately higher (+32 percent) with wall-mounted seats than floor-mounted seats.

5. The body elements, steel plate elements, and longitudinal chassis cap elements had slightly higher (+4 percent) to very substantially higher (+278 percent) maximum stresses with wall-mounted than floor-mounted seats.

6. The maximum stresses in the perimeter frame elements and the lateral chassis elements were slightly lower (−9 percent) to substantially lower (−77 percent) with wall-mounted seats than floor-mounted seats.

The following general conclusions can be drawn about the responses of typical medium-duty transit buses to bus deceleration:

1. With full seat belt use, maximum member stresses should in general be lower than with staggered seat belt use or front seat belt use only. The more-uniform distribution of passenger inertial loads resulting from full seat belt use offers a clear advantage to the structure of the transit bus under bus deceleration.

2. Maximum member stresses should in general be lower with floor-mounted than wall-mounted seats. With their exterior legs attached directly to the perimeter of the frame, floor-mounted seats appear to offer a distinct benefit to the bus structure under bus deceleration.

3. The maximum stresses could be relatively high in the seat anchorage members with wall-mounted seats and in the perimeter frame members with floor-mounted seats. Thus, these members could yield at relatively low levels of deceleration and could continue to yield and deform as deceleration increases. In this way, the authors believe that these secondary structural members may act as "passenger shock absorbers" in that their deformation (and hence their absorption of energy) could cushion the passengers, thus reducing the level of deceleration felt by the passengers.

4. In general, the differences should be relatively small between the maximum member stresses for shorter medium-duty transit buses and the corresponding maximum stresses for longer buses. Although the shorter buses have fewer passengers and thus less passenger inertial load, the longer buses have more members and provide more avenues for stress redistribution, which results in lower member stresses per unit of load. It should again be noted, however, that the inertia of the bus members and the bus components was not included in the analyses. Therefore, the inertia of the additional 4 ft of bus in the 25-ft bus versus the 21-ft bus could cause more maximum member stresses to be higher in the 25-ft bus under actual bus decelerations.

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# Relationship Between Truck Accidents and Highway Geometric Design: A Poisson Regression Approach

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A Poisson regression model is proposed to establish empirical relationships between truck accidents and key highway geometric design variables. For a particular road section, the number of trucks involved in accidents over 1 year was assumed to be Poisson-distributed. The Poisson rate was related to the road section's geometric, traffic, and other explanatory variables (or covariates) by a loglinear function, which ensures that the rate is always nonnegative. The primary data source used was the Highway Safety Information System (HSIS), administered by FHWA. Highway geometric and traffic data for rural Interstate highways and the associated truck accidents in one HSIS state from 1985 to 1987 were used to illustrate the proposed model. The maximum likelihood method was used to estimate the model coefficients. The final model suggested that annual average daily traffic per lane, horizontal curvature, and vertical grade were significantly correlated with truck accident involvement rate but that shoulder width had comparably less correlation. Goodness-of-fit test statistics indicated that extra variation (or overdispersion) existed in the developed Poisson model, which was most likely due to the uncertainties in truck exposure data and omitted variables in the model. This suggests that better quality in truck exposure data and additional covariates could probably improve the current model. Subsequent analyses suggested, however, that this overdispersion did not change the conclusions about the relationships between truck accidents and the examined geometric and traffic variables.

The passage of the 1982 Surface Transportation Assistance Act preempted more-restrictive state vehicle size and weight limits and has allowed longer and wider trucks to travel on a designated national highway network (*1*). Furthermore, states are expected to provide reasonable access beyond the national network to truck terminals and service facilities. Safety questions, such as whether the current highway design is adequate to serve these larger trucks and which highway geometric conditions pose the most serious safety problems for large trucks, are of primary concern. These questions can be better addressed if truck accident involvement rates, defined as the number of trucks involved in highway accidents per truck miles traveled, can be accurately estimated for different truck types under different highway geometric conditions.

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The occurrences of vehicle accidents have long been recognized as complex events involving the interactions of many factors. Previous attempts to establish relationships between vehicle accidents and highway geometric design variables have had mixed results, and no specific relationships are widely accepted. In addition, the relationships have typically been studied through conventional linear regression models. These models are, however, known to have several undesirable statistical properties in describing discrete random events such as vehicle accidents. (A brief review of relevant studies is given later.) The objective of this paper is to present a potential statistical framework for establishing relationships between truck accidents and key highway geometric design variables. The types of trucks of interest are large trucks with gross vehicle weight ratings of 10,000 lb or more. The specific truck safety questions that this proposed model framework is intended to address include:

1. Given a section of highway, how safe is it for large trucks in terms of accident involvement rate and accident probability?
2. Given a set of highway geometric design elements, which elements are relatively more critical to the safety performance of large trucks?
3. What reduction in large-truck accident involvement rates can be expected from various improvements in highway geometric design?

In this study, accident probability refers to the probability of observing $y$ vehicles involved in accidents during a period of time, where $y = 0, 1, 2, 3, \ldots$.

This paper is organized as follows: statistical models used in recent studies to establish relationships between vehicle accidents and highway geometric design are briefly reviewed; a statistical framework for establishing such relationships is presented; and the data used in this study and the associated statistics are described. Then the results are summarized and directions for future work are suggested.

## LITERATURE REVIEW

The empirical relationships between vehicle accidents and highway geometric design variables, such as horizontal curvature, vertical grade, lane width, and shoulder width, have been addressed in many studies. For example, *NCHRP Report 197* contains a summary of research performed until 1978, including a synopsis of major findings from more than 400 reports and publications (*2*). More recent studies include those by Zegeer et al. (*3*) and Joshua and Garber (*4,5*). Unfortunately, most of the studies did not distinguish accident rates between trucks and other types of vehicles. Among those studies that did focus on truck accidents, accident rates were often reported at the aggregated national or state level for a particular roadway class, not in terms of specific highway geometries (*1,6*). Joshua and Garber's study is, to the best of our knowledge, the first study that actually focused on quantifying the effect of highway geometric design variables on truck accidents statistically. At present, we are unaware of any statistical investigations that have established a relationship by both truck configuration and accident severity type. An up-to-date literature review on large-truck accidents and their relationships to various roadway, driver, vehicle, traffic, and environmental characteristics as well as safety implications of various truck configurations relevant to highway geometric design, is contained in a paper by Miaou et al. (*7*).

The need to establish relationships between truck accidents and geometric design and the frustration among researchers to find such relationships were properly described by Harwood et al. (*8*):

> The data . . . clearly illustrate the effect of two key variables related to hazardous materials routing—roadway type and area type—on truck accident rate. An attempt was made to determine the relationship between two traffic volume factors (AADT [annual average daily traffic] and percent trucks) and truck accident rate, but no consistent results were obtained. Consideration of the effects of additional geometric variables . . . on truck accident rates . . . would be desirable. . . . However, it should be recognized that the development of reliable relationships between geometric features and accidents is a difficult statistical task. Previous attempts . . . have had mixed results and no set of geometric-accident relationships is widely accepted.

Indeed, vehicle accidents are complex processes involving the interactions of many factors, including not only the road, but also the vehicle, the drivers, the traffic, and the environment (e.g., weather and lighting conditions). To establish such a relationship, the analysis requires good accident, traffic, and highway geometric data, as well as good truck travel (or exposure) information. In part, lack of accurate information on vehicle miles traveled by truck configuration and inadequate accident reporting have made the evaluation of truck safety performance on national highways nearly unattainable. A TRB Special Report was prepared specifically to address these data problems and has proposed a national monitoring system (NMS) for truck safety (*9*). Unfortunately, the data collection plan adopted in NMS did not address the highway geometric aspect of truck safety issues.

The rest of this section presents a discussion on statistical models that have been used for establishing the relationships between vehicle accidents and highway geometric design variables. We then summarize the characteristics of the problems and outline the desired capabilities of a candidate model for establishing such relationships.

**Vehicle Accident Models**

Multiple linear regression models have been used frequently in establishing vehicle accidents–geometric design relationships, as summarized in *NCHRP Report 197* (*2*). The regression models have the following general form:

$$\frac{y_i}{v_i} = x_i'\beta + \varepsilon_i \qquad (i = 1, 2, \ldots, n) \tag{1}$$

where

- $i$ = road section index;
- $y_i$ = number of vehicles involved in accidents during a time period;
- $v_i$ = total vehicle miles traveled;
- $x_i$ = vector of explanatory variables (or covariates) associated with the section, such as AADT, horizontal curvature, vertical grade, and shoulder width;
- $\beta$ = vector of regression coefficients to be estimated; and
- $\varepsilon_i$ = zero mean model residual for road section $i$.

Vehicle miles traveled, $v_i$, is usually estimated as 365 × $\text{AADT}_i$ × $\ell_i$ × (number of years under consideration), where $\text{AADT}_i$ and $\ell_i$ are the AADT (in number of vehicles) and the section length (in miles) of section $i$, respectively. It was also suggested that the dependent variable $y_i/v_i$ be log-transformed whenever appropriate. The ordinary least-squares (OLS) method was typically used to estimate the regression coefficients, although the weighted least-squares (WLS) method was occasionally used. A recent study that used the multiple linear regression model was reported by Zegeer et al. (*3*), in which the WLS method was used for coefficient estimation. For the 10,900 curved road sections they studied, 12, 123 vehicle accidents were reported over a 5-year period. However, those accidents occurred on only 44.3 percent of the road sections, so 55.7 percent of the road sections had no observed accidents in the 5 years.

There are several statistical properties of the conventional multiple linear regressions that are considered undesirable in establishing the relationships between vehicle accidents and highway geometric design. These undesirable properties relate mainly to the underlying distributional assumption of a conventional multiple linear regression model, some of which have been discussed by Jovanis and Chang (*10*). The following are some examples:

- For a given road section, the number of vehicles involved in accidents are random discrete events that take nonnegative integer values: 0, 1, 2, 3, . . ., each of which has some probability of being observed. The use of a continuous distribution such as normal distribution to model accident events is at best an approximation to a truly discrete process. Furthermore, the occurrences of vehicle accidents are sporadic in nature. In most studies of this kind, the analyst is faced with a problem of dealing with a large number of road sections that have no accidents during the observed period. Zegeer et al.'s study is a good example (*3*). This suggests that for several years most of the road sections considered would have a much higher probability of being observed with no accidents than with more than one accident. In other words, the underlying distribution of the occurrences of vehicle accidents on most of the road sections is positively (or rightly) skewed. Normal distribution is not a good approximation under this condition.

- There are other inferential assumptions of multiple linear regression in Equation 1 that are probably too restrictive for this type of study—for example, the residuals of the model, $\varepsilon_i$, are assumed to be uncorrelated with the explanatory variables, $x_i$. Other limitations of using Equation 1 include the following: (a) it may occasionally predict negative accident involvement rates, and (b) it does not provide a clear linkage between accident involvement rate and accident probability. That is, for an estimated accident involvement rate from the regression model, it is difficult to compute the probability of observing $y$ vehicles involved in accidents on a particular road section during a period of time.

In contrast to multiple linear regression models, the Poisson regression models are widely used for modeling accident and mortality data in epidemiology. It is only in a recent study by Joshua and Garber (*4,5*) that the model was introduced to establish the relationships between truck accidents and highway geometric design. A limitation of using the Poisson regression model, which is well-known in the statistical literature (*11,12*), is that the variance of the data is restrained to be equal to the mean. In this study, we used a Poisson regression model with a different model structure than that used by Joshua and Garber and considered the consequences of the limitation on Poisson distribution.

Although the proposed model in this paper is also in the Poisson context, it does not have two limitations that we have observed for the Joshua and Garber model. We will briefly discuss these limitations using their first model [Equation 28 in the work by Joshua and Garber (*4*)] as an example. First, one can show that the Joshua and Garber model will always give a small prediction of truck accidents for relatively leveled highway sections no matter what other variables are being included in the model. In other words, regardless of the AADT and the percentage of trucks for a given road section, as long as that road section is relatively flat, the predicted number of accidents for that section based on their model will be small. Second, it can also be shown that the Joshua and Garber model suggests that increases in AADT or length for a given road section, while holding other variables constant, will lead to a decrease in the predicted truck accident rate for that road section. This is contrary to what one would expect.

### Model Capabilities

The characteristics of the problem a researcher will face in establishing empirical relationships between vehicle accidents and highway geometric design can be summarized as follows:

- Vehicle accidents are complex interactions involving many factors. Many of these variables will never be available for individual road sections. Therefore, in developing empirical models, one should recognize the fact that, no matter how many covariates one manages to include, there are always some variables that will be excluded, especially those qualitative types of variables.
- The occurrences of vehicle accidents are sporadic and discrete random events.
- Road sections differ not only in geometric features and traffic conditions but also in vehicle exposure.
- Vehicle accidents and exposure data are both subject to sampling and nonsampling errors. Not all accidents are reported, especially minor property damage accidents. Also, vehicle exposure data come primarily from FHWA's Highway Performance Monitoring System (HPMS) (*13*), which is a sampling-based system.

These characteristics suggest that a potential model for establishing such relationships for trucks should be a probabilistic model capable of

- Addressing safety questions in terms of both accident involvement rate and accident probability,
- Predicting "nonnegative" accident involvement rates,
- Taking into account the differences in truck exposure across road sections,
- Giving proper statistical weights to a great portion of road sections with no observed truck accidents,
- Providing inferential statistics that allow the evaluation of model uncertainties due to the uncertainties of truck exposure data and possible omitted variables in the model, and
- Handling different roadway classes, truck configurations, and accident severity types.

## POISSON REGRESSION MODEL

### Model Formulation

Consider a set of $n$ highway sections of a particular roadway type, say, rural Interstate. Let $Y_i$ be a random variable representing the number of trucks involved in accidents on highway section $i$ during a period of, say, 1 year. Furthermore, assume that the amount of truck travel (or exposure) on this highway section, $V_i$, is also a random variable, estimated through a highway sampling system, such as HPMS. Associated with each highway section $i$, there is a $k \times 1$ covariate vector, denoted by $x_i = (x_{i1} = 1, x_{i2}, \ldots, x_{ik})'$, describing its geometric characteristics, traffic conditions, and other relevant attributes. Given $V_i$ and $x_i$, truck accident involvements $Y_i$, $i = 1, 2, \ldots, n$ are postulated to be independent, and each is Poisson-distributed as

$$p(Y_i = y_i | \Lambda_i = \lambda_i, V_i = v_i, x_i) = \frac{(\lambda_i v_i)^{y_i} e^{-\lambda_i v_i}}{y_i!}$$

$$(i = 1, 2, \ldots, n; y_i = 0, 1, 2, \ldots,) \qquad (2)$$

where $\Lambda_i$ ($>0$) is the truck accident involvement rate on highway section $i$, and it is expected to vary from one highway section to another, depending on its covariates $x_i$. For each highway section $i$, the Poisson model implies that the conditional mean is equal to the conditional variance:

$$E(Y_i|\Lambda_i = \lambda_i, V_i = v_i, x_i) = \mathrm{Var}(Y_i|\Lambda_i = \lambda_i, V_i = v_i, x_i) = \lambda_i v_i \qquad (3)$$

and is proportional to truck exposure $v_i$ for a given truck accident involvement rate $\lambda_i$. The definition and properties of the Poisson process are well-known and will not be repeated here [see elsewhere (*14*)].

To establish a relationship between truck accident involvement rate and highway geometric and traffic variables, the following exponential form is used:

$$\Lambda_i = \exp(x_i'\beta + \varepsilon_i) = \lambda_i \exp(\varepsilon_i) \tag{4}$$

where $\beta$ is a $k \times 1$ coefficient vector and $\varepsilon_i$ is a specification error due, for instance, to omitted variables. (Note that higher-order and interaction terms of covariates can be included in Equation 4 without difficulties whenever appropriate.) This particular loglinear relationship ensures that the truck accident involvement rate is always nonnegative. Also, the specification error, $\varepsilon_i$, admits the fact that the functional relationship is at best an approximation to the true relationship. This type of functional relationship has been widely employed in statistical literature and found to be very flexible in fitting different types of count data (*12,14–16*).

If $x_i$ and $V_i$ are given with no (or negligible) uncertainties and $\Lambda_i$ is assumed to be a constant (i.e., $\varepsilon_i = 0$, for all $i$), then Equation 2 becomes a classical Poisson regression model. The uncertainties in $V_i$ and $\Lambda_i$ introduce extra variations (or overdispersion) in the Poisson model (*12,17*). The consequences of ignoring the extra variations in the Poisson regression are that the maximum likelihood estimates (MLEs) of the regression coefficients, $\beta$, under the classical Poisson model, are still consistent; however, the variances of the estimated coefficients would tend to be underestimated. In other words, we may overstate the significance levels of the estimated coefficients (*11,18*).

Throughout this study, we used the classical Poisson regression, assuming that truck exposure $V_i$ and covariates $x_i$ were observed without error and that truck accident involvement rate, $\Lambda_i$, was a constant for each road section $i$. The potential underestimation of coefficient variance, because of overdispersion in the Poisson regression model, was corrected using an estimate of overdispersion suggested by Wedderburn (*19*) [and elsewhere (*20*)].

### Model Estimation and Diagnostic Checking

Regression coefficients, $\beta$, were estimated using the MLE procedure. The detailed derivation of the MLEs and the corresponding covariance matrix is omitted from this paper but can be found in work by Miaou et al. (*7*). The MLE was obtained by maximizing the log-likelihood function, $L(\beta)$, with respect to the coefficient $\beta$ using a nonlinear optimization technique called the Davidon-Fletcher-Powell algorithm (*21*). In determining whether a specific variable should be included in Equation 4, we first checked to see if the estimated coefficient of the variable had the expected sign, and then we examined whether its $t$-statistic was greater than 1.96 (or 1.645 for a lower $\alpha$-level). In addition, we used Akaike's information criterion (AIC) for model selection. Models with smaller AIC values are preferred. Bozdogan's article is an excellent reference on the theory and application of AIC criterion (*22*).

To help assess the overall goodness-of-fit of the proposed model, we considered two statistics: Pearson's chi-square statistic ($X^2$) and likelihood ratio statistic ($G^2$) (*23*). The basic idea of both statistics is to compare the observed frequency with the expected frequency based on the model. In this particular study, the observed and the expected frequencies refer to the observed and the expected number of trucks involved in accidents [$y_i$ and $\exp(x_i'\hat{\beta})v_i$] on each road section, respectively. However, $X^2$- and $G^2$-statistics are usually poorly approximated by chi-square distribution when a large number of $y_i$ are zero (*23*).

To reconcile this small frequency problem, instead of comparing frequencies for each individual road section, we consider a group of road sections as a comparing unit. First, each covariate is categorized into a number of subintervals; road sections are then cross-classified according to the values of their covariates. In other words, we generate a multidimensional "contingency table" in such a way that each road section is assigned to one of the cells in the table according to the values of their covariates. For road sections that are assigned to the same cell, their observed and expected numbers of trucks involved in accidents and truck exposure are added up respectively to produce the corresponding cell values. Note that a cell with no realized truck exposure is called a structural zero and is treated as if it does not exist.

To construct the multidimensional table, the strategy used in this study to choose the cutoff points of each covariate was to avoid creating too many cells with very low estimated frequencies. The $X^2$- and $G^2$-statistics were then computed by comparing the observed and the expected frequencies in each cell of the multidimensional table. The estimated model may be inadequate when $X^2$ and $G^2$ are greater than a reference point, $\chi^2_{0.05}$ (df $= H - p$), where df is the degrees of freedom, $H$ is the total number of cells with truck exposure greater than zero, and $p$ is the number of coefficients considered in the model. However, it should be noted that there are many possible reasons for a model to fail the tests, including the overdispersion problem discussed earlier.

In sum, a selected model should have (a) the expected signs in all estimated coefficients, (b) low AIC value and (c) high $t$-statistics for model coefficients. For the model to be useful in practice, the signs of the coefficients were given the highest consideration. At the same time, covariates with signs in coefficients contrary to expectation should be checked and further investigated. If higher-order terms in the model are found to be statistically significant, they should be considered but carefully checked.

## DATA AND SUMMARY STATISTICS

### Data Source

Data from the Highway Safety Information System (HSIS), an accident data base developed by the Highway Safety Research Center (HSRC) of the University of North Carolina for FHWA, were employed for developing relationships between truck accidents and key highway geometric design variables. The HSIS currently contains information on five states. Of these states, one in the Midwest was considered to be the state that has the most complete information on highway geometric design. In addition, this state was also the only HSIS state with a "historical" road inventory file in which year-to-year changes on highway geometric design are recorded. Thus, accidents in a given year can be matched to the road inventory information of the same period. For these reasons, road and accident data from this HSIS state were chosen for illustration in this paper.

Detailed descriptions of the HSIS data base, including data quality, are available in a guidebook prepared by the HSRC and in work by Miaou et al. (*7*). At the time of this study, the data base of this state is maintained on an annual basis from 1985 to 1987. Data are stored in six files: roadlog, horizontal curvature, vertical grade, accident, vehicle, and occupant files. Thus, these files had to be linked before any analysis could be performed. Key variables used in linking these files were the route numbers and milepoints at which accidents occurred and the route numbers and milepoints at which a road section, a curve, and a grade began and ended.

Each record on the road inventory file represented a homogeneous section in terms of its road characteristics, such as number of lanes, lane width, shoulder width, median type and width, and AADT. Thus, for example, once the lane width changes on a particular road section, this section, along with its neighboring sections, was redelineated to reflect the change. It should be noticed that each road section in the roadlog file was not necessarily homogeneous in terms of its horizontal curvature and vertical grade. On the other hand, each road section in the horizontal curvature and vertical grade files was homogeneous in terms of its horizontal curvature and vertical grade, respectively, but not necessarily in terms of other road characteristics.

Rural Interstate highways and the associated large-truck accidents from 1985 to 1987 were extracted to illustrate the proposed model. Horizontal curvature and vertical grade files were only available for 1987. Since these two highway geometric elements usually change very little over the years, we used the 1987 horizontal curvature and vertical grade data for all 3 years under consideration.

### Data and Statistics

The time period considered in this study was 1 year, which means that the same road section, even if nothing had changed, was considered as three independent sections—one for each year from 1985 to 1987. Therefore, there were 1,644 road sections of rural Interstate, which constituted 8,779.21 lane-mi of roadway, during the 3-year period. Data for each year contained roughly one-third of the total sections and lane miles. The section lengths varied from 0.01 to 14.9 mi—with an average of 1.35 mi. Simple descriptive statistics of these 1,644 road sections and the associated truck accidents, truck exposure, traffic, and key highway geometric design variables are given in Table 1. The following is a detailed discussion of these data.

#### *Accident Data*

During the 3-year period, 933 large trucks were involved in accidents on the rural Interstate highways, regardless of truck configuration and accident severity type. With the total estimated to be 1,057.54 million truck-mi (MTM), the overall truck accident involvement rate was 0.88 truck involvements per MTM. Out of these 933 large trucks, the relative splits by single-unit and combination trucks were 109 and 824, respectively. These accidents occurred on only 32.2 percent of the road sections. The maximum number of trucks involved in accidents being observed on an individual road section was 10. On average, each section in the rural Interstate had 0.57 trucks involved in accidents per year.

#### *Truck Exposure Data*

For each highway section, AADT, truck percentage, and section length were available from the roadlog file. For each road section $i$, truck exposure was computed as $v_i$ = 365

**TABLE 1 Mean and Standard Deviations of Characteristics of 1,644 and 5,105 Road Sections**

| Accidents, Exposure, and Covariates | Measure | 1,644 Sections Mean | 1,644 Sections Standard Deviation | 5,105 Sections Mean | 5,105 Sections Standard Deviation |
|---|---|---|---|---|---|
| No. of Trucks Involved in Accidents | | 0.5675 | 1.0854 | 0.1832 | 0.5585 |
| Truck Exposure (million truck miles) | | 0.6433 | 0.8019 | 0.2072 | 0.3514 |
| AADT/Lane (in 1000's of vehicles) | | 1.8943 | 1.6509 | 1.6640 | 1.3774 |
| Horizontal Curvature (degrees/100 ft arc) | CD | | | 1.0108 | 2.0655 |
| | CCR | 4.5664 | 33.1867 | | |
| | MAC | 0.4409 | 0.7530 | | |
| | MC | 1.1484 | 1.9144 | | |
| Vertical Grade (percent) | VG | | | 2.1736 | 1.6039 |
| | GCR | 2.7305 | 5.1859 | | |
| | MAG | 1.9623 | 1.1955 | | |
| | MG | 2.5262 | 1.5027 | | |
| Shoulder Width: Inside & Outside (ft) | | 13.7518 | 1.0930 | 13.7877 | 1.0103 |
| Section Length (miles) | | 1.3489 | 1.5226 | 0.4343 | 0.6916 |

$\times$ AADT$_i$ $\times$ $(T\%_i/100)$ $\times$ $\ell_i$, where $T\%_i$ is the percentage of trucks (e.g., 15) and $\ell_i$ is the length of road section $i$. Note that AADT$_i$ $\times$ $(T\%_i/100)$ is the "truck AADT" of road section $i$. Thus, truck exposure is related to truck AADT and length of the road section. As indicated in Equation 3, the proposed Poisson model has the property that the expected number of trucks involved in accidents on a road section is proportional to the truck exposure of that road section.

### *Traffic Variable*

AADT is typically used to indicate traffic condition or congestion level of a road section. Because the number of lanes varies from one road section to another, particularly in urban areas, in this study we generalized this variable by considering AADT per lane. This traffic variable represents the average density of vehicle flow on the road in an average day. Conceptually, the higher the vehicle density, the greater the chance for a truck to be involved in a conflicting position with other vehicles when negotiating its way through the road section. For example, consider two road sections, $i$ and $j$, with identical geometric design and the same truck exposure (i.e., $v_i = v_j$), but Section $i$ is more congested than Section $j$: $(\text{AADT/lane})_i > (\text{AADT/lane})_j$; one would expect to observe more trucks involved in accidents on Section $i$ than that on Section $j$. (Of course, this example is valid only if all other conditions, such as environment and driver factors, are the same between the two road sections.)

### *Highway Geometric Design Variables*

The highway geometric design variables available from the selected HSIS state included (a) lane width, (b) paved shoulder width, (c) median width and type, (d) horizontal curvature, and (e) vertical grade. Because all of the road sections were coded as having 12-ft lane width, we were unable to distinguish the effects of different lane widths on truck accident involvement rate in this study. In addition, most road sections were divided. The highway geometric design variables used in the model follow.

**Shoulder Width** Paved shoulder widths were recorded separately for inside (or left) and outside (or right) shoulders in a given direction. In this study, because the inside and outside shoulder widths are highly correlated, we considered total shoulder width (i.e., the sum of inside and outside shoulder widths). Furthermore, we considered 20 ft to be an "ideal" shoulder width in that it practically adds an additional lane on each side of the road. Based on this consideration, we defined a variable called "deviation from ideal shoulder width," which is the total shoulder width short of the ideal shoulder width. In other words, for a particular road section $i$ deviation from ideal shoulder width, denoted by SWD$_i$, was defined as SWD$_i$ = max$\{0, 20 - \text{SW}_i\}$, where SW$_i$ is the total inside and outside shoulder width of Section $i$.

**Horizontal Curvature and Vertical Grade** Horizontal curvatures and vertical grades were coded in degrees per 100 ft arc and percent, respectively. In addition, positive values indicated "right turn" and "upgrade" whereas negative values indicated "left turn" and "downgrade." As indicated earlier, each road section in the road inventory file was relatively homogeneous in terms of general road characteristics and traffic conditions, but not necessarily in horizontal curvature and vertical grade. Therefore, each road section in the road inventory file may have contained more than one horizontal curvature or vertical grade. Two ways of resolving this problem were considered. One way was to create surrogate measures to characterize the curvature and grade conditions along the length of a road section. Another way was to disaggregate those road sections with multiple curvatures and grades into smaller subsections in such a way that each subsection contains a unique set of horizontal curvature and vertical grade. The former was considered less direct from the engineering point of view and it may be difficult for design engineers to incorporate these measures into their current practice, but the second method was considerably easier to interpret in a design context. However, it should be mentioned that because the location of an accident is often estimated and occasionally it is roughly assigned to the nearest milepost of the route on which it occurred, assigning vehicle accidents to very short road sections is more susceptible to locational error than assigning to longer road sections. In this study, we used both approaches for comparison purposes.

Three surrogate measures for horizontal curvature and three surrogate measures for vertical grade were devised to characterize the horizontal and vertical alignments of each road section. On a particular road section $i$ with length $\ell_i$ (in miles), assume that along the length of the section there are $K$ curved subsections associated with it, indexed by $k = 1, 2, \ldots, K$. Each subsection $k$ has length $\ell_{i,k}$ and curvature $\theta_{i,k}$ (which could be zero, positive, or negative). Similarly, we assumed that there were $G$ different vertical graded subsections associated with a road section $i$, and each subsection had length $\ell_{i,g}$ and grade $\omega_{i,g}$ (which again could be zero, positive, or negative), where $g = 1, 2, \ldots, G$. These surrogate measures for a section $i$ were defined as follows:

1. Horizontal curvature change rate (CCR) and vertical grade change rate (GCR):

$$\text{CCR}_i = \sum_{k=1}^{K-1} |\theta_{i,k+1} - \theta_{i,k}|$$

$$\text{GCR}_i = \sum_{g=1}^{G-1} |\omega_{i,g+1} - \omega_{i,g}| \tag{5}$$

If there is only one curvature (or grade), CCR (or GCR) is defined as zero.

2. Mean absolute horizontal curvature (MAC) and mean absolute vertical grade (MAG):

$$\text{MAC}_i = \left(\sum_{k=1}^{K} \ell_{i,k}|\theta_{i,k}|\right) \Big/ \ell_i$$

$$\text{MAG}_i = \left(\sum_{g=1}^{G} \ell_{i,g}|\omega_{i,g}|\right) \Big/ \ell_i \tag{6}$$

3. Maximum absolute horizontal curvature (MC) and maximum absolute vertical grade (MG):

$$MC_i = \max\{|\theta_{i,1}|,|\theta_{i,2}|, \ldots, |\theta_{i,K}|\}$$

$$MG_i = \max\{|\omega_{i,1}|,|\omega_{i,2}|, \ldots, |\omega_{i,G}|\} \quad (7)$$

All of these surrogate curvature measures are in the unit of degrees per 100 ft arc, and surrogate grade measures in percent. Similar measures were used by Joshua and Garber (*4*). Note that these surrogate measures are not unique (i.e., different combinations of curves and grades can result in the same values).

The second way to resolve the multiple curvatures and grades problem was to disaggregate road sections into smaller subsections so that each section had a unique set of curvature and grade measures. On the basis of this approach, 1,644 road sections were disaggregated into 5,105 subsections. Each subsection contained unique curvature and grade information. Two additional trucks involved in accidents were included because of the difference in assigning truck accidents to road sections when accidents occurred right at the cutoff point of two neighboring sections. With these redefined subsections, truck accidents occurred on 12.8 percent of the sections. In other words, 87.2 percent of the redefined road sections had no observed truck accidents during a year. Simple statistics for the redefined road sections are also given in Table 1. In our analysis, we used the absolute value of horizontal curvature (CD) and vertical grade (VG) on each subsection as the covariates.

## RESULTS

### Model Estimation and Selection

The proposed Poisson regression model was applied to develop empirical relationships between truck accidents and key highway geometric design variables described in the last section. The considered covariates include the following:

- $x_{i1}$ = 1, representing a dummy intercept;
- $x_{i2}$ = AADT per lane (thousands of vehicles);
- $x_{i3}$ = horizontal curvature (degrees/100-ft arc);
- $x_{i4}$ = vertical grade (%); and
- $x_{i5}$ = deviation from ideal shoulder width (ft).

Several models were tested using different horizontal curvature and vertical grade measures. Table 2 presents the estimated coefficients (using maximum likelihood method) and their asymptotic *t*-statistics, AIC values, and negative log-likelihood functions $[-L(\hat{\beta})]$ of the tested models. Asymp-

**TABLE 2 Estimated Coefficients of Tested Poisson Regression Models and Associated Statistics**

| Covariates & Statistics | Measure | Model 1 | Model 2 | Model 3 | Model 4 |
|---|---|---|---|---|---|
| $x_{i1}$ = 1 (Dummy Intercept) | | -14.4889 (-53.15) | -14.6804 (-55.96) | -14.6413 (-54.38) | -14.6833 (-55.67) |
| $x_{i2}$ = AADT/Lane (in 1000's of vehicles) | | 0.069864 (3.77) | 0.037828 (2.01) | 0.076083 (4.14) | 0.044691 (2.41) |
| $x_{i3}$ = Horizontal Curvature (degrees/100 ft arc) | CD | | | | 0.172513 (8.96) |
| | CCR | 0.000699 (0.42) | | | |
| | MAC | | 0.217495 (5.07) | | |
| | MC | | | 0.046771 (3.59) | |
| $x_{i4}$ = Vertical Grade (percent) | VG | | | | 0.162218 (7.09) |
| | GCR | 0.001249 (0.21) | | | |
| | MAG | | 0.222829 (7.91) | | |
| | MG | | | 0.091138 (3.70) | |
| $x_{i5}$ = Deviation from Ideal Shoulder Width (ft) | | 0.064703 (1.48) | 0.020024 (0.48) | 0.036062 (0.83) | 0.038589 (0.93) |
| $-L(\hat{\beta})$ | | 1542.78 | 1492.57 | 1521.31 | 2222.95 |
| *AIC* Value | | 3095.56 | 2995.14 | 3052.62 | 4455.90 |
| Predicted vs. Observed Total Truck Involvements | | 932.35 933.00 | 932.88 933.00 | 932.99 933.00 | 931.85 935.00 |
| Number of Road Sections | | 1644 | 1644 | 1644 | 5105 |

*Values in parentheses are t-statistics of the coefficients above.*

totic $t$-statistics were computed without the adjustment of possible overdispersion described earlier. Models 1, 2, and 3 used surrogate measures (i.e., CCR and GCR, MAC and MAG, MC and MG) for road sections with multiple horizontal curvatures and vertical grades, and Model 4 used the data for the disaggregated subsections that had a unique set of curvature and grade measures (i.e., CD and VG). Note that in Table 2, AIC values and log-likelihood functions are not comparable between Models 1 through 3 and Model 4 because the number of road sections (or sample sizes) used in developing the models was different.

Among the set of three surrogate measures for horizontal curvature and vertical grade, mean absolute curvature and vertical grade, MAC and MAG, (Model 2) performed better than the other two measures (Models 1 and 3) in terms of their ability to explain truck accident variations, which was indicated by lower negative log-likelihood function and AIC values. In addition, the coefficients estimated for MAC and MAG were in agreement with those obtained with CD and VG (Model 4) in terms of their values and signs. This suggests that surrogate measures MAC and MAG are probably more appropriate for establishing the model when disaggregating road sections into smaller subsections is not desirable. Overall, the estimated regression coefficients of AADT per lane, horizontal curvature, and vertical grade were all found to be highly significant in terms of their asymptotic $t$-statistics (when compared with a 1 percent $\alpha$-level). Deviation from ideal shoulder width, on the other hand, was found to be insignificant at a 5 percent $\alpha$-level.

### Goodness-of-Fit Test and Adjusted $t$-Statistic

Model 4 was selected to conduct further goodness-of-fit tests. To compute the $X^2$- and $G^2$-statistics discussed earlier, road sections were first categorized by their covariates. In other words, multidimensional contingency tables cross-classified by the covariates were constructed. However, these two statistics were quite sensitive to the way covariates were divided into subintervals. Specifically, different categorization of the covariates could result in different contingency tables that had very different $X^2$- and $G^2$-statistics. We have examined, by trial and error, several categorizations of the covariates that were identified to be statistically significant in the selected model for generating the multidimensional contingency tables. An interesting feature we have found was that the generated tables were always very unsymmetrical, that is, the observed accident involvements concentrated on a small number of cells whereas the rest of the cells had very few observations. This was because most of the road sections had similar geometric features—for example, a great percentage of road sections were straight sections (i.e., curvature = 0) and a significant proportion of road sections had 2 percent grades. After trying several possible combinations of categorizing the covariates, we selected a contingency table that appeared to be most reasonable and did not contain too many cells with estimated frequency less than 1. The selected table had 112 cells in total, in which 9 cells had no realized truck travel and were ignored.

The $X^2$- and $G^2$-statistics were 160.35 and 150.33, respectively. The model failed the chi-square test at a 5 percent $\alpha$-level, that is, $X^2$ and $G^2$ are greater than $\chi^2_{0.05}(\text{df} = 103 - 5 = 98) = 122.3$. This suggests that the overdispersion problem, due most likely to the uncertainties in truck exposure data and the omitted variables in Equation 4, is quite significant and that the current model can probably be improved by including additional covariates and by improving the accuracy of truck exposure data.

To check how the overdispersion affected the conclusions reached in the last subsection, the overdispersion parameter ($\tau$) was estimated to be $X^2/(H - p)$, as suggested in Wedderburn (*19*). Better estimates of the $t$-statistics were derived by dividing the $t$-statistics obtained from the Poisson regression model by $\tau^{1/2}$ (= 1.28) (*23*,p.457). This adjustment did reduce the significance levels of the regression coefficients, but it did not alter the conclusions about the relationships between truck accidents and the examined geometric and traffic variables.

### Example

To give an example of how truck accident involvement rate and accident probability can be computed from Model 4 for a rural Interstate highway section in the selected HSIS state, let us consider a hypothetical road section with the following characteristics:

- Lane width: 12 ft
- Section length: 1 mi
- Number of lanes: 4
- AADT/lane: 3,000 vehicles per lane
- Percentage trucks: 20
- Horizontal curvature: 3 degrees per 100-ft arc
- Vertical grade: 2 percent
- Shoulder width (left + right): 14 ft

Truck exposure in a year is first computed as

$$\begin{aligned} v &= 365 \times (\text{number of lanes} \times \text{AADT per lane}) \\ &\quad \times (\text{percentage trucks}/100) \times (\text{section length}) \\ &= 365 \times (4 \times 3{,}000) \times (20/100) \times 1 \\ &= 876{,}000 \text{ truck-mi} \end{aligned}$$

Based on the estimated model, the truck accident involvement rate is estimated as

$$\begin{aligned} \lambda &= \exp\{-14.6833 + 0.044691 \times 3 + 0.172513 \times 3 \\ &\quad + 0.162218 \times 2 + 0.038589 \times (20 - 14)\} \\ &= \exp\{-13.475718\} = 1.4047 \times 10^{-6} \end{aligned}$$

The expected number of trucks involved in accidents on this road section in 1 year is estimated at $E(y) = \lambda v = 1.4047 \times 10^{-6} \times 876{,}000 \approx 1.23$ trucks. The probability of observing $y$ trucks involved in accidents on this particular road section in 1 year is then $p(y) = [(1.23)^y \exp(-1.23)]/y!$. For example, the probability of observing two trucks involved in accidents is $p(y = 2) = [(1.23)^2 \exp(-1.23)]/2! = 0.22$.

## SUMMARY AND FUTURE RESEARCH

The Poisson regression model was proposed for establishing empirical relationships between truck accidents and key highway geometric design variables. Highway geometric and traffic data for rural Interstate highways and the associated truck accidents in one of the HSIS states from 1985 to 1987 were used to illustrate the proposed model. The estimated model suggested that AADT per lane, horizontal curvature, and vertical grade are significantly correlated with truck accident involvement rate, but that shoulder width has comparably less correlation. Goodness-of-fit test statistics indicated that extra variations (or overdispersion) existed in the developed Poisson model, which was most likely due to the uncertainties in truck exposure data and omitted variables in the model. This suggests that better quality in truck exposure data and additional covariates could probably improve the current model. The effect of correcting for the overdispersion was found to lower the significance level of the estimated Poisson regression coefficients. It, however, did not change the conclusions about the relationships between truck accidents and the examined traffic and highway geometric design variables.

An immediate extension of this study would be to apply the proposed Poisson regression model to other roadway types and to consider truck accidents by truck configuration and accident severity type. It would also be of practical interest to quantify the respective contribution of truck exposure data uncertainty and omitted variables to the overall overdispersion in the model. Other discrete distributions, such as negative binomial distribution, should also be explored. Finally, it would be interesting to study the sensitivity of the developed models to the uncertainties of accident location information.

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# Characteristics of Commercial Vehicle Drivers in Ontario

JULIUS GORYS AND GREG LITTLE

Considerable attention has been paid recently to the drivers of commercial vehicles, in light of concerns about the impact of deregulation. In 1988 a major on-highway survey of commercial vehicles was carried out in the province of Ontario. Data in the survey were also collected on the characteristics of truck drivers, including age, the means and terms of employment, driving record, and extent of experience and training. Information from the survey provides useful insight as to the working conditions of drivers and the nature of the drivers themselves.

Drivers of large commercial vehicles have been the focus of public attention for many years. With trucking deregulation in the United States in 1980 and in Canada in 1988, its impact on these drivers has been the subject of concerns, specifically the following:

- In their drive to become more profitable, firms have changed the security, remuneration, and conditions of employment of drivers, resulting in less driver accountability.
- There is suspicion that deregulation will encourage the deferring of vehicle maintenance in order to reduce costs.
- As a result, safety has been compromised with a deterioration in driver and vehicle performance, especially because of increased traffic congestion and the economic need to increase vehicle use.

The correlation between economic regulation and safety performance is unclear, given that the level of truck safety was controversial even before deregulation. Indeed, some have argued that the principal safety problem was not necessarily the degree of economic regulation but the adequacy of enforcement (*1*).

A review of truck driver–related characteristics was considered appropriate in the context of an overall goods movement study, given this and the fact that truck drivers have one of the highest occupational fatality rates (*2*,p.431). Concerns have also been raised because of a perceived shortage of qualified drivers that was identified in the 1980s. Considerable goods traffic was diverted from the rail mode to the truck mode; at the same time, working conditions and pay levels in the trucking industry were not sufficient to attract or retain truck drivers (*3,4*).

The opportunity to research these issues arose during the undertaking of the 1988 Ontario Commercial Vehicle Survey. For that survey, several questions were asked about employment status, length of driving experience, driver remuneration and training, and union affiliation. An attempt was made to collect driver license numbers in order to analyze driving records.

The intent of this paper is to report on these findings. It should be noted that although driver-related information was but a small part of the information collected during the survey, it is an initial step in building a base of knowledge from which other parts of the ministry can analyze further, develop policies, and refine enforcement procedures.

The literature contains many examples of studies that are more specific in their analysis or application, for example, studies that evaluate driver improvement programs, reasons and exposure rates for truck accidents by driver and vehicle type, and the effects of deregulation. This study does not attempt to draw on or compare itself to all this research.

## SURVEY LOGISTICS

The Ontario Ministry of Transport periodically carries out on-highway surveys of intercity truck activity. Its 1988 Ontario Commercial Vehicle Survey was conducted over 23 weeks between March and November 1988 at 57 locations, mainly at vehicle inspection stations and border crossings (Figure 1). At most locations, surveying was done over a full 24-hr period, weather permitting.

A total of 19,225 commercial vehicle drivers were given a 29-question interview that lasted between 8 and 12 min; 8 questions specifically pertained to the driver. The survey total represented an overall sample rate of 18 percent. The survey was conducted by trained university students and contract personnel, not vehicle inspection staff. Vehicle inspection staff directed traffic into the inspection station and weighed the vehicle. There was no threat, implied or actual, for nonresponse; drivers who refused to participate were immediately free to proceed. Enforcement was practiced during the survey with respect to weight or safety violations, but there was little evidence of station avoidance.

Generally, between two and five vehicles were directed into the inspection area, where they were weighed and the drivers interviewed. All other vehicles were allowed to bypass the inspection area until the surveying was completed. The process was then repeated. There was no attempt to bias the survey by class of truck or carrier; the likelihood of being selected was equal.

Drivers responded favorably to the survey. There was a refusal rate of 3.9 percent for the entire survey. For each individual question, there was a further nonresponse rate on

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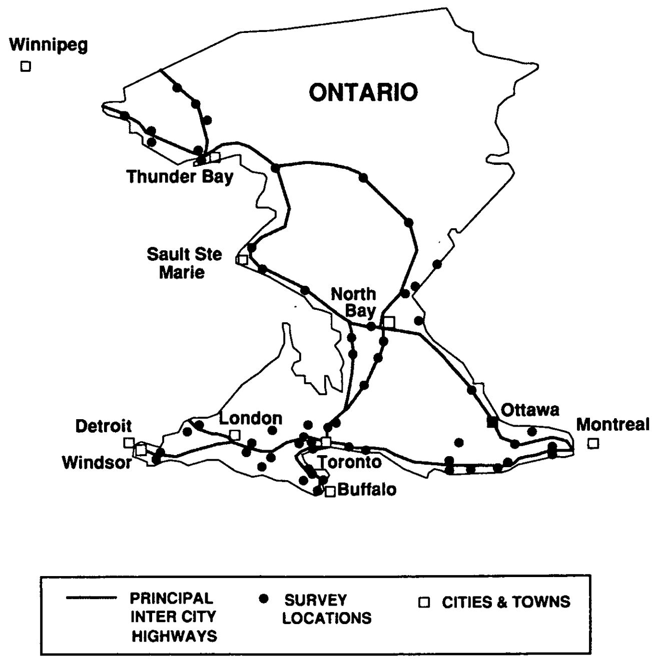


FIGURE 1 Locations of survey stations.

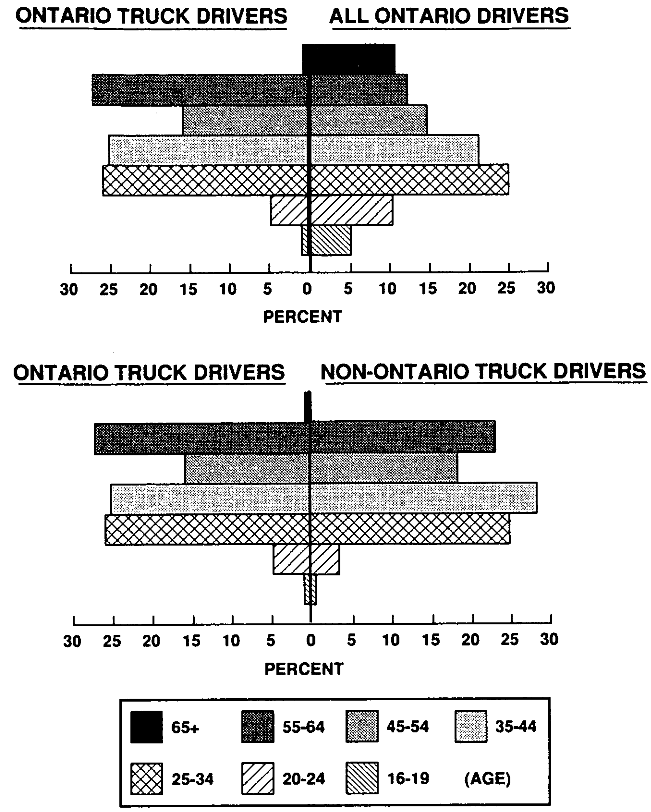


FIGURE 2 Age profile of men drivers.

the order of 9 to 15 percent. Ontario drivers, who constituted 81 percent of all drivers surveyed, were also asked if they would voluntarily provide their driver license number as part of the interview. About 8,800, or 64 percent, of the surveyed Ontario drivers provided a valid license number for analysis. This figure roughly approximated a 1 percent sample of all Ontario truck drivers. Their records were later compared with a representative subset of the licensed Ontario driving population.

The survey results were controlled at the analysis level so as to not overrepresent the same driver on the same trip but at a different location. Given the expanse of the province, a driver or vehicle could conceivably pass as many as nine inspection stations on a single trip.

## OTHER SURVEY FINDINGS

On the order of 76 percent of the vehicles surveyed were tractor and semi-trailer units, 16 percent were straight trucks, 6 percent were tractor and two-trailer units, and 2 percent were other types. The three most prominent body styles of vehicles were van or box (61 percent), flatbed (14 percent), and tanker (8 percent). Fifty-eight percent of the vehicles had five axles, 20 percent had six or more axles, and 22 percent had less than five axles.

Fifty-two percent of the movements surveyed were by private carriers (companies that haul their own goods), in contrast to 48 percent by for-hire carriers (companies who haul goods for other firms). Forty-four percent of all movements and 51 percent of all weight carried crossed the provincial border. The mean trip length of the first truck-trailer unit was 560 km (350 mi).

## AGE AND SEX PROFILE

The truck driving profession is dominated by men; fewer than 1 percent of drivers surveyed were women. The demographic profile of drivers is principally occupied by those in the advanced age categories, if the over-65 age category is excluded (Figure 2).

There were only modest differences among the commercial vehicle driver categories (Table 1). The mean age of drivers was 39 years. This was slightly younger than that—41 years—found in a recent (1988) Florida survey of 900 intercity drivers conducted at 16 inspection stations (*5,6*). About one-third of all drivers were in their thirties. Most differences were found in the under-30 age category, partly because one must be 18 years old in Ontario to drive a truck (American standards restrict truck drivers to 21 years and older).

A more profound difference was discovered when comparing company drivers to brokers and owner operators, individuals who own a truck and operate it themselves, usually under contract with a carrier. Sixteen percent of brokers and owner operators were under 30 years old compared with 20 percent of company drivers.

## EMPLOYMENT STATUS

One of the most visible outcomes of deregulation has been that firms more often contract work out to driver services, particularly to owner operators. Greater use of such drivers

**TABLE 1 Percentage of Commercial Vehicle Drivers by Age Category**

| AGE GROUP (years) | BROKERS & OWNER OPERATORS | COMPANY DRIVERS | FOR-HIRE DRIVERS | PRIVATE DRIVERS | ONTARIO DRIVERS | NON-ONTARIO DRIVERS | TOTAL |
|---|---|---|---|---|---|---|---|
| 16-19 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 |
| 20-29 | 15.4 | 20.4 | 20.1 | 18.8 | 20.1 | 18.1 | 19.7 |
| 30-39 | 36.9 | 34.6 | 35.3 | 34.8 | 34.9 | 34.3 | 34.8 |
| 40-49 | 29.7 | 25.8 | 26.1 | 26.8 | 25.7 | 30.1 | 26.5 |
| 50-59 | 14.6 | 15.2 | 14.7 | 15.7 | 15.3 | 14.6 | 15.1 |
| 60-64 | 2.0 | 2.3 | 2.2 | 2.2 | 2.3 | 1.8 | 2.2 |
| 65 + | 0.4 | 0.7 | 0.6 | 0.7 | 0.8 | 0.3 | 0.7 |
| Total | 100.0 | 100.0 | 100.0 | 100.0 | 100.0 | 100.0 | 100.0 |
| Mean Age (yrs) | 39.3 | 38.9 | 38.8 | 39.0 | 38.9 | 39.1 | 39.0 |

**TABLE 2 Driver Employment by Type of Carrier**

| EMPLOYMENT CATEGORY | FOR-HIRE CARRIER | PRIVATE CARRIER | COMBINED |
|---|---|---|---|
| Company Driver | 61.8% | 83.7% | 73.1% |
| Broker/Owner Operator | 28.5% | 8.6% | 18.3% |
| Agency Driver | 7.5% | 5.7% | 6.5% |
| Self Employed | 2.2% | 2.0% | 2.1% |
| TOTAL | 100.0% | 100.0% | 100.0% |

is a function of

- Attempts by firms to reduce both fleet size, direct driver employment levels, and associated benefits; and
- Opportunities for individual drivers to attain higher pay by operating independently rather than as part of a larger firm.

The 1988 survey established that a quarter of all drivers operated outside of a traditional firm. The largest proportion, 18 percent, was made up of brokers or owner operators, almost three times that for agency drivers (Table 2). The aforementioned Florida survey found greater evidence of owner operator use—27 percent. For-hire carriers relied on agency drivers and brokers to a far greater extent (36 percent) than private carriers (14 percent).

In contrast, the 1983 Ontario Commercial Vehicle Survey identified that 79 percent were company drivers, 9 percent were brokers or owner operators, 8 percent were self-employed, and 4 percent were agency drivers. For-hire carriers were also found to be three times as likely to contract work out to brokers or owner operators (7,p.141). Undoubtedly, pressures to become more competitive induced greater use of owner operators during the period 1983–1988.

About one-third of all drivers surveyed belonged to a work-related association, mainly a union, and mainly the Teamsters union. Even among brokers and owner operators there was considerable union membership (Figure 3).

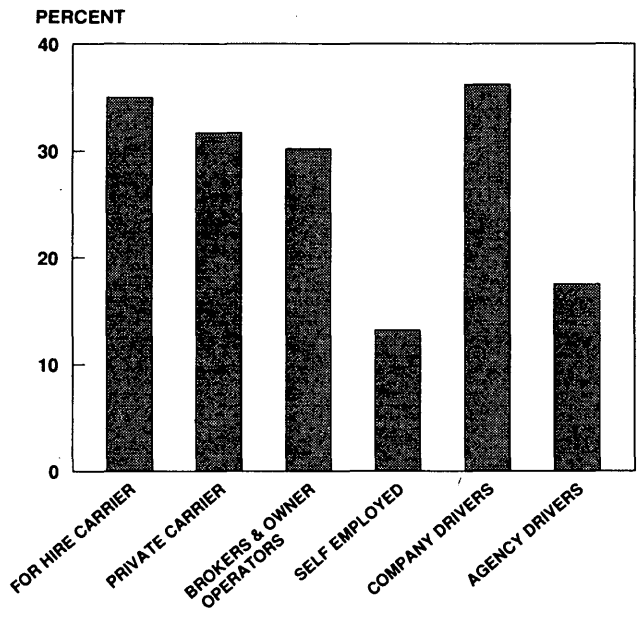


**FIGURE 3 Levels of membership in work-related associations.**

Unionization levels are generally attributed to be higher in Canada than in the United States. This was borne out in an examination of international trips: 30.7 percent of Ontario drivers involved in international (cross-border) trips reported being members of a union, compared with only 21.6 percent of U.S.-based drivers on similar trips. The Florida study found only 10 percent union membership among drivers but identified that another 27 percent formerly belonged to a union.

**TABLE 3 Years of Commercial Driving Experience**

| DRIVER TYPE | YEARS OF EXPERIENCE | | | | | |
|---|---|---|---|---|---|---|
| | UNDER 9 | 10-19 | 20-29 | 30 & OVER | TOTAL | MEAN |
| Owner Operator | 27.3% | 37.2% | 22.9% | 12.6% | 100.0% | 16.2 |
| Company Driver | 30.7% | 34.4% | 21.4% | 13.5% | 100.0% | 15.4 |
| For-Hire Carrier | 30.6% | 34.7% | 21.7% | 13.0% | 100.0% | 15.8 |
| Private Carrier | 29.2% | 36.2% | 21.2% | 13.4% | 100.0% | 16.0 |
| Ontario Carrier | 30.1% | 35.0% | 21.5% | 13.5% | 100.0% | 15.9 |
| Other Carrier | 29.8% | 37.1% | 21.4% | 11.7% | 100.0% | 15.6 |
| Straight Trucks | 47.1% | 29.7% | 14.9% | 8.3% | 100.0% | 12.2 |
| Tractor & Semis | 27.0% | 36.3% | 22.5% | 14.2% | 100.0% | 16.6 |
| Work Assoc'n | 20.0% | 35.6% | 25.4% | 19.0% | 100.0% | 18.6 |
| Non Work Assoc'n | 35.0% | 35.3% | 19.4% | 10.3% | 100.0% | 14.5 |
| Average | 30.0% | 35.4% | 21.5% | 13.1% | 100.0% | 15.9 |

## DRIVING EXPERIENCE

The survey found that the typical driver had spent 16 years driving a commercial vehicle, comparable to the Florida study (15.7 years), and that some 35 percent of drivers had at least 20 years of experience. There were modest differences in experience among the different classes of drivers:

- Brokers and owner operators and those hauling for private carriers had slightly more years of experience than did company drivers and those hauling for private carriers (Table 3).
- Those hauling for Ontario and non-Ontario carriers had comparable levels of experience.
- There was a more profound difference in years of experience (19 years) between drivers who belonged to a work-related association (e.g., union) and drivers that did not (15 years), particularly among drivers with more than 30 years of experience.
- There was an equally notable difference in experience between drivers of smaller straight trucks and drivers of larger tractor and semi-trailer units. A significantly higher proportion of straight truck drivers had less than 9 years of experience than did any other category of driver.

When experience in years is compared to age, a significant pattern is revealed. There is a considerable drop-off in the number of drivers with few years of experience in the 40-plus age groups. This tends to support the perception that truck driving is an attractive occupation only to younger people; relatively few enter the profession in their later working years. This has implications for the future supply of drivers, given the high mean age of drivers and the general aging of the North American population.

## DRIVING REMUNERATION

About 33 percent of drivers surveyed indicated that they were paid on the basis of time, and 31 percent noted that they were paid on the basis of distance traveled (Table 4). Payment based exclusively on weight transported was unusual, although many drivers were paid by a combination of methods, including weight transported.

For-hire carriers generally paid their drivers on the basis of distance traveled; private carriers, on the basis of time. Drivers of particular truck body styles were compensated in different fashions for their services. For example, drivers of dump trucks, concrete mixers, and flatbed units were most often paid on the basis of time; for drivers of float units and car carriers, pay was based on distance; for drivers of hopper units and tankers, pay was based on a combination of methods.

By contrast, the 1987 Florida survey ascertained that 60 percent of the drivers were paid on the basis of distance trav-

**TABLE 4 Method of Driver Payment**

| PAYMENT TYPE | FOR-HIRE | PRIVATE | COMBINED |
|---|---|---|---|
| Time (Hourly/Salaried) | 20.7% | 44.2% | 32.9% |
| Distance travelled | 39.0% | 23.7% | 31.0% |
| Weight/Volume/Commodity type | 8.3% | 4.5% | 6.3% |
| Combination of methods | 32.0% | 27.6% | 29.8% |
| Total | 100.0% | 100.0% | 100.0% |

eled, 33 percent were paid on the basis of characteristics of the load, and only 7 percent on the basis of a salary or hourly wage (*8*). Private carriers had a higher proportion of drivers paid on the basis of time—13 percent, versus 5 percent for for-hire carriers—but it was nowhere near that of the Ontario experience. (This question was not repeated for the 1988 version of the Florida survey.)

The concern about method of driver payment is that it almost represents a direct economic incentive to drive longer hours and violate hours-of-service regulations designed and instituted to enhance safety. The lack of rest that results from such situations has been found to translate into a greater risk of truck driver-related crashes (*9*,p.30).

## DRIVER TRAINING

Driving a commercial vehicle is a demanding occupation. To continue to meet those demands it is advantageous to upgrade one's skills through additional training. It was asked whether, during the past few years, the drivers had taken a course that either taught or enhanced their knowledge of first aid, provided superior defensive driving skills, or trained and upgraded them in using dangerous goods and mitigating spills.

It should be noted that just taking such courses does not necessarily mean that one is a safer driver. There are variations in program length, depth of coverage, and year last taken. Indeed, some researchers have concluded that support for "common sense notions" in the traffic safety literature with respect to driver improvement activity programs is far from unequivocal (*10*).

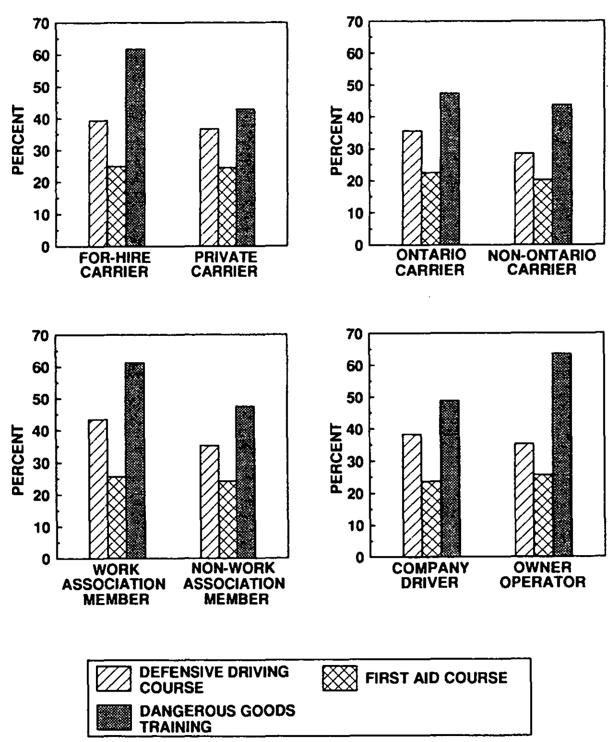


**FIGURE 4 Extent of driver training.**

Sixty-two percent of the drivers indicated that they undertook some form of enhanced training. The dangerous goods training course, a requirement by law for the transport of such goods, had a 47 percent attendance level; 34 percent attended a defensive driving course and 22 percent took a first aid course. About 11 percent of the drivers had taken all three course types. Drivers belonging to a work related-association or employed by a for-hire firm or Ontario-registered carrier were more likely to have taken such courses (Figure 4).

## DRIVING RECORD

The public has a negative image of the commercial truck driver that is reinforced with every accident or incident involving trucks. It was thus considered opportune to investigate the driving records of commercial vehicle drivers to ascertain whether the public perception of those drivers was justified.

It was not possible to access the records of non-Ontario drivers, hence the request for driver license numbers was asked only of Ontario drivers. About 8,000 valid Ontario driver license numbers were obtained, a response rate of 64 percent. Because the overwhelming majority of drivers were male, the comparative analysis concentrated only on male driving records.

The driving records of these individuals were contrasted with the male proportion of a subset of 2.1 million drivers in the entire file of 6.5 million Ontario drivers. It should be noted that the comparison constitutes the total driving record of truck drivers, not just that obtained while driving a truck.

The following results were not formally checked for bias; it is acknowledged that the data may be biased insofar as poor drivers or those driving under suspension may have declined to provide their license numbers, although it was stated that the information would be kept in confidence and would not be used against them. Further analysis is intended to identify and correct those biases. Therefore, these results should be viewed with caution. Driving records are updated each year. Driver violations remain on the driver's record for 3 years after the offense.

It was found that the overall driving records of Ontario's truck drivers were superior to those of the general driving population (Table 5) in all categories except one—collisions. The data are to be analyzed further to ascertain whether geographical or other considerations (e.g., at-fault information) were associated with this statistic.

The driving records have not been adjusted to reflect distance traveled; given the vehicle kilometers traveled by truck drivers, their record would appear even more favorable than illustrated here. Alternatively, some enforcement staff believe that the court system tends to be more lenient with truckers than with other drivers because driving is a trucker's livelihood.

A review of the driving records of commercial vehicle drivers revealed that the proportion of offenses declines as age increases (Figure 5), similar to the experience of all younger drivers. This relationship appears to be consistent with other information. A U.S. analysis of fatal accident involvement rates by driver age for large trucks also illustrated that younger drivers tended to be over involved not only in fatal truck accidents but in nearly all other conditions (*11*).

**TABLE 5 Comparison of Driving Records**

| CATEGORY | TRUCK DRIVERS | ALL MALE DRIVERS |
|---|---|---|
| Demerit Point Incidence | 27.8% | 35.7% |
| Convictions | 31.1% | 35.9% |
| Speeding Violations | 19.9% | 25.4% |
| Collisions | 9.0% | 6.8% |
| Alcoholic Collisions | 0.1% | 0.3% |
| Suspensions | 2.8% | 5.7% |
| Warning Letters | 5.0% | 7.5% |
| Interviews | 1.2% | 1.4% |

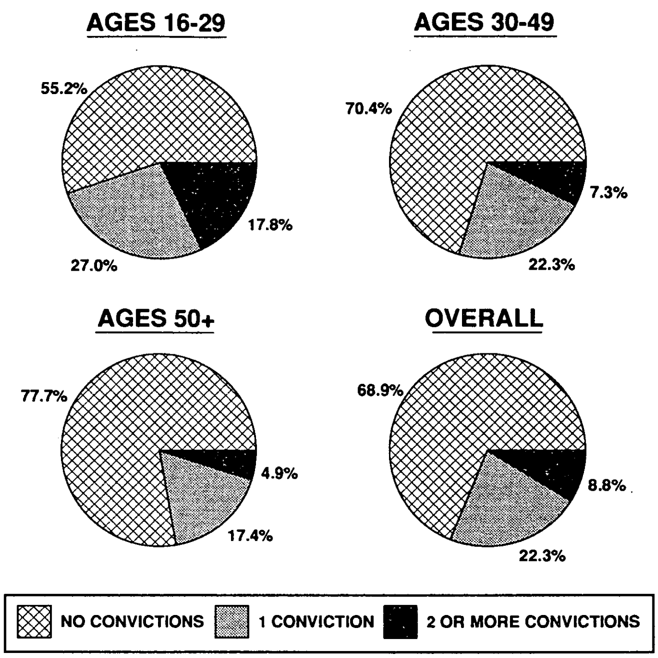


**FIGURE 5 Total traffic convictions for Ontario commercial vehicle drivers (men only), 1988 (*Source: 1988 Ontario Commercial Vehicle Survey*).**

## HOURS OF SERVICE

In response to concerns about safety, legislation has been passed that regulates the number of hours that commercial vehicle drivers can work daily, weekly, or biweekly. The 1988 Commercial Vehicle Survey requested that drivers provide information on how many hours they would work that day and how much of their time would actually be spent driving.

An examination of the mean hours worked and driven revealed only modest differences between subpopulations of drivers (i.e., for-hire versus private). Generally, on average, intercity drivers worked 10 hr that day, of which 7 hr were actually spent in transit.

Not surprisingly, intracity goods movement studies in Toronto and Ottawa established that far less time was spent in transit for urban goods movement trips; because of shorter distances and smaller shipment sizes, more time was spent loading and unloading. In the Toronto study, it was found that 4 hr each was spent driving, loading, and unloading; in the Ottawa study, 3.8 hr was spent driving (42 percent) and 5.3 hr (58 percent) was spent loading and unloading (*12,13*).

Sixty-three percent of intercity drivers worked between 9 and 13 hr that day; 9 percent of drivers surveyed worked more than 13 hr that day. Contracted drivers and firms tended to work longer hours than their counterparts (Figure 6).

Of interest is the number of truck drivers who work more than 13 hr and those who drive for a large proportion of that time. Several studies have identified that truck driver performance tends to deteriorate as hours of driving increase (*14*,p.vi) and that elevated levels of accident risk have been associated with driving after more than 12 hr on duty and driving during early morning hours (*9*,p.29).

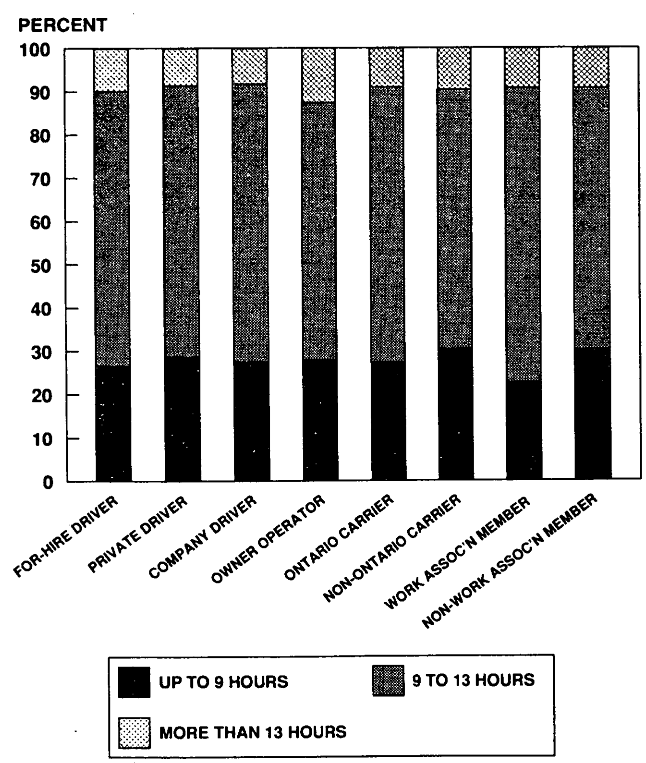


**FIGURE 6 Hours of service.**

**TABLE 6 Ontario Accident Levels, 1985–1989 (*24*)**

| YEAR | ALL ACCIDENTS | ALL FATAL ACCIDENTS | TRUCK ACCIDENTS | FATAL ACCIDENTS |
|---|---|---|---|---|
| 1985 | 189,750 | 1,036 | 60,386 | 417 |
| 1986 | 187,286 | 951 | 62,895 | 416 |
| 1987 | 203,431 | 1,085 | 71,172 | 483 |
| 1988 | 228,398 | 1,076 | 67,653 | 471 |
| 1989 | 247,038 | 1,106 | 70,510 | 466 |

## DISCUSSION OF RESULTS

The purpose of the 1988 Ontario Commercial Vehicle Survey was to provide information on the characteristics of intercity truck movements in the province. Driver characteristics collected and reported here represent an initial step toward the provision of time series statistics on commercial vehicle drivers for policy development and analysis purposes. It is of great use insofar as it provides background information on several issues and concerns.

For example, hours-of-service regulations in the United States are more restrictive than those in Ontario. Recent enforcement blitzes have caught many Canadian truckers on cross-border trips in violation of those regulations (*15*).

The data collected on hours of service can be useful for evaluating whether current regulations are appropriate. They can be measured against reported driving practices, American regulatory experiences, and concerns about overworked drivers (*16*,p.1;*17*,p.22), as well as whether they can be adequately articulated by drivers. For instance, a study of drivers of heavy freight trucks in the Netherlands found that hours-of-work regulations were frequently contravened, probably because they were poorly understood (*14*).

In addition, data on methods of driver payment, hours-of-service restrictions, and work-related association membership provide insight into issues affecting owner operators and unions (*18*), overall employment levels in the industry, and tax levels, affording the ability to better understand and deal with confrontational situations such as recent highway and border blockades held by independent Canadian truckers (*19*). These data also confirm that if current trends continue, there could be an employment deficiency in the industry that will be exacerbated by projected growth in the trucking sector in the 1990s. One study estimated that 34,000 new tractor-trailer drivers will be required in the Canadian trucking industry during this period (*20*). This deficiency is occurring when it is becoming increasingly difficult to attract new drivers from its traditional blue-collar sources and at the same time retain existing drivers because of associated working conditions and income levels.

It is hoped that these situations can be corrected with a redirection of educational resources and recruitment-related actions by other government agencies, and that drivers, carriers, shippers, and government can work together to alleviate other concerns.

Since driver-related violations as opposed to vehicle deficiencies are more often cited as causes of truck accidents (*21*), tying accident statistics to this information allows the findings of other studies to be confirmed or refuted. For example, one such study states that younger drivers and longer hours of driving were associated with the higher crash involvement of large trucks (*22*).

Overall, truck-related accidents have been increasing at a slower rate than all accidents (by 17 to 30 percent, respectively) in Ontario over the past 5 years (Table 6); however, fatal truck accidents have increased more (12 percent) than all fatal accidents (7 percent).

Further analysis of the information collected enables a comparison of driver and vehicle statistics from the survey with accident information and the development of vehicle kilometer exposure rates by vehicle style with a view to updating policy and enforcement practices.

The data also provide a starting point to evaluate the success of driver training programs, given that the level of driver training is one of the most commonly cited factors associated with heavy-vehicle accidents (*23*). It was noted in another study that drivers with formal training were more likely to have accidents than those without (*6*).

It is reasonable to expect that when the next major provincial on-highway survey is undertaken in 1993, the list of questions asked of drivers themselves would be continued and perhaps augmented. The commercial vehicle surveys were not developed to evaluate the impact of deregulation, but a 1993 survey would represent a useful postderegulation view of some of the effects of that action, one to be compared with the 1988 information that approximates a view of deregulation in transition. Unfortunately, there is little substantive information to approximate the prederegulation view in Ontario.

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*The opinions expressed in the document are solely those of the authors and in no way reflect the position of the Ontario Ministry of Transport and Communication.*

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*Abridgment*

# Countermeasures for Truck Accidents on Urban Freeways: A Review of Experiences

KAY FITZPATRICK, DAN MIDDLETON, AND DEBBIE JASEK

Because of the rise in truck volume, the interaction of these large vehicles with other traffic, and the publicity given to major truck accidents, public awareness of the consequences of truck accidents and incidents is heightened. A literature review, telephone interviews, and visits to selected sites provided information on several truck accident countermeasures implemented on high-volume urban freeways. An FHWA survey found that 15 states have restricted trucks to certain lanes to improve highway operations. The New Jersey Turnpike and I-5 north of Los Angeles have sections on which trucks are restricted to a separated facility. Ramp treatments include reconstruction to remove outside curbs, installation of tall barriers, evaluation of the appropriateness of posted ramp speeds, and active and passive warning signs. Truck diversions or bans exist in Minneapolis–St. Paul, south of Cincinnati, San Diego, Los Angeles, and Atlanta. Allowing trucks to park in a park-and-ride lot during nighttime hours and increasing enforcement to restrict the length of stay in inappropriate locations (e.g., mainlane shoulders or along entrance and exit ramps) are measures used to reduce shoulder parking. Maryland, Virginia, and California have urban truck inspection stations, and Chicago, Tampa, and Seattle have elements of their incident management program directed toward trucks.

The issue of truck accidents and incidents on urban freeways is a vital concern for both traffic managers and the general public. Public awareness is now heightened because of the rise in truck volume, the interaction of these large vehicles with other traffic, and the publicity given to major truck accidents. Besides the fatalities and injuries resulting from truck-involved accidents, the excessive costs and delays caused by these accidents and by incidents have prompted several operating agencies to consider strategies to reduce the problem.

A literature review, telephone interviews, and visits to selected sites identified countermeasures used to reduce truck accidents on urban freeways. The countermeasures include lane restrictions, separate truck facilities, ramp treatments, truck diversions, truck bans, reduction of shoulder parking, urban truck inspection stations, and incident management. This paper contains a summary of identified experiences and issues associated with these countermeasures.

## LANE RESTRICTIONS

Several states are restricting the lanes in which trucks can operate. The objective in restricting trucks to the right lane or lanes is typically to improve highway operations and reduce accidents. To assess the effect of lane restrictions for trucks, FHWA in 1986 asked its division offices to report experiences with lane restrictions (*1*). The FHWA survey indicated that 15 states implemented restrictions to improve highway operations. While the benefit of improved highway operations is reduced accidents, only eight states reported that their truck restrictions were directed toward reducing accidents. The field survey also indicated that, in most cases, restrictions were applied without detailed evaluation plans including before and after studies.

Information on accident experience with lane restrictions in Florida was included in the FHWA survey (*1*). In 1988 Florida conducted a 6-month experiment to determine the effect of prohibiting large trucks from using the left lane on I-95. With signs posted about every mile—and good media coverage and strict police enforcement—98 percent compliance was achieved. The accident rate for all vehicles decreased 2.5 percent for an all-day (24-hr) period but increased 6.3 percent during the prohibition period (7:00 a.m. to 7:00 p.m.). The proportion of accidents involving trucks with three or more axles decreased 3.3 percent during the hours of the restriction.

The Virginia Department of Transportation (DOT) instituted a lane restriction for trucks on its I-95 section of the Washington, D.C., Capital Beltway between I-395 and the Woodrow Wilson Bridge (near the Virginia state line) on December 1, 1984. A lane restriction was similarly imposed by Maryland on its portion of the Beltway in an attempt to reduce accidents. Several studies of the Virginia I-95 data evaluated accidents, speeds, and volumes to determine the effects of the countermeasure. Initial studies recommended the retention of the countermeasure primarily because of favorable public perception and the decrease (or no change) in accident severity, even though accident rates had risen (reasons were not provided to explain the increase in accident rates) (*2,3*). Later studies revealed increased accident rates (*4*; unpublished data, Virginia DOT, 1989), so the removal of the truck lane restriction was recommended.

## SEPARATE TRUCK FACILITIES

The New Jersey Turnpike, which is approximately 120 mi long, has a 33-mi segment that consists of interior (automobile) lanes and exterior (truck/bus/car) lanes within the same right-of-way. For 23 mi, the interior and exterior roadways in each direction have three lanes. On the 10-mi section that opened in November 1990, the exterior roadway has two lanes and the interior roadway has three lanes per direction. Each roadway has 12-ft lanes and 12-ft shoulders. Directional flows

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are separated by a concrete median barrier, and the inner and outer flows are separated by a metal beam guardrail. Trucks and buses are restricted to the outer roadway, but smaller vehicles can use either the inner lanes or outer lanes. The current mix of automobile traffic is approximately 60 percent on the inner roadways and 40 percent on the outer roadways.

In California the reconstruction of a section of I-5 north of Los Angeles resulted in two parallel roadways. After completion of the new interstate roadway, the old roadway was maintained to carry truck traffic. The three major interchanges of I-5 with I-405, Route 210, and Route 14 span about 4 mi and are designed to accommodate heavy traffic demands. For example, I-5 at Route 14 carries an average daily traffic of 122,000; 13.5 percent of this volume is trucks. The truck bypass lanes at I-405 are relatively short, but the truck facility is continuous between Routes 210 and 14, using the old roadway.

Truck facilities have been considered for the corridor connecting the San Pedro ports and downtown Los Angeles, the I-10 Houston-Beaumont (Texas) corridor, and the Houston North Freeway (I-45). For the facility in Los Angeles, proposals include using the paved Los Angeles River channel as an exclusive truck facility (*5*), and using the Alameda Street corridor to carry trucks and trains within a right-of-way also shared by automobiles. Studies on potential sites in Texas concluded that the construction of exclusive truck facilities was not warranted because of limited truck volumes along certain sections of the corridor and the estimated cost of the facility (*6,7*).

## RAMP TREATMENTS

Restrictive geometry on freeway ramps, resulting in a compromise of safety for large-truck operations, has become a concern for many agencies. Ramps can be especially difficult for large trucks to negotiate when inadequate design elements such as insufficient superelevation, tight curvature, unanticipated changes in compound curves, and short acceleration and deceleration lanes are combined with inappropriate posted advisory speeds.

Two ramps in Detroit, Michigan, were improved to reduce truck accidents by adding a tall (72-in.) reinforced concrete barrier intended to contain overturning trucks and their loads. One ramp connects westbound I-94 to southbound I-75, and the other ramp serves I-75 northeastbound traffic desiring to stay on I-75 northbound at the I-375 interchange. The I-75 two-lane ramp, originally constructed with an outside curb, was also reconstructed by adding a "wedge" of pavement to cover the outside curb, and the superelevation was increased to 0.074 ft/ft across both lanes and the shoulder.

Maryland and Virginia reevaluated ramp speeds on the Capital Beltway to determine whether the posted speeds were appropriate for trucks. Virginia reduced speeds on 44 ramps (*4*) and Maryland also reduced speeds on several ramps. California is evaluating turning roadways to determine the adequacy of speed signing for trucks.

Passive signs are sometimes used to describe the ramp alignment and to warn of the potential for truck rollover. For example, a truck-tipping sign is used on southbound I-95 approaching the Capital Beltway in Maryland. In some cases, flashing wigwags are added to signs to increase conspicuity. Active signs are used on ramps in Atlanta, Georgia, to inform drivers that their speeds are excessive. Any vehicle exiting the freeway that exceeds the design speed of the ramp will cause yellow warning lights to flash in a wigwag fashion. Because these devices do not discriminate between cars and trucks, they flash almost continuously.

Ongoing research sponsored by the Insurance Institute for Highway Safety and FHWA will evaluate the effectiveness of flashing warning lights (wigwags) in reducing truck speeds on ramps. In these studies, the devices are activated only by trucks, in contrast to signs used in Atlanta, which respond to both cars and trucks traveling faster than the preset speed.

## TRUCK DIVERSIONS

In Minneapolis–St. Paul, traffic signs encourage truck traffic to divert to the bypass rather than travel straight through the central business district area on more congested freeways. This action seeks voluntary compliance and is not a regulatory ban. Although the effects of this countermeasure have not been studied, local officials do not believe that any significant diversion has resulted.

A fiery truck accident on the I-71/75 segment in Covington, Ky. (south of Cincinnati), resulted in the imposition of a truck diversion order by the Kentucky governor on July 8, 1986. Trucks were diverted from northbound I-71/75 to I-275, a freeway bypass around Cincinnati. The diversion order was expected to shift accidents from the interior interstate highways to I-275 with no net change in accidents for the entire region. However, for the I-71/75 segment, the diversion was expected to reduce truck-involved accidents by approximately 9 percent (*8*).

## TRUCK BANS

In an effort to reduce congestion, San Diego has restricted trucks from Route 163 through scenic Balboa Park. The merging of traffic from five to two lanes, a 6 percent grade, and a lack of acceleration and deceleration lanes for interchanges all contribute to heavy congestion on the freeway. Public opinion prohibits construction of additional lanes because of the extensive landscaping and scenic location of the freeway (*9*).

A truck ban currently exists on the Ventura Freeway in Los Angeles primarily because the facility, which opened in 1940, has a pavement that is too weak to support trucks. Officials from the California Department of Transportation (Caltrans) report that with no trucks, this 7-in. pavement is still in good condition. The only large vehicles allowed on the freeway are transit buses. There is also a truck avoidance policy currently in effect for the Harbor Freeway (I-710) in Los Angeles during major reconstruction. It is only a voluntary ban, and Caltrans reports that the reduction in trucks is negligible.

Beginning in December 1978, a new truck restriction required that through trucks approaching Atlanta use the I-285 bypass instead of the freeways that run through the center of the city. To evaluate compliance with this ban, a survey conducted by the Georgia Department of Transportation on March 25, 1980, showed a violation rate of 5.4 percent.

## REDUCTION OF SHOULDER PARKING

The reduction of nonemergency shoulder parking assumes that if shoulders are used by motorists for emergency stopping only, a reduction of certain types of accidents could result. Agent and Pigman found that although the number of all accidents on Kentucky limited-access highways involving vehicles on shoulders was small (1.8 percent), the number of fatal accidents involving vehicles on shoulders was significant (11.1 percent) (*10*). Tractor trailers were overrepresented in shoulder accidents when compared with their involvement in all accidents.

Maryland has given truck drivers an alternative to shoulder parking by allowing trucks to park in a park-and-ride lot during the nighttime hours. Michigan DOT observed an increasing trend on I-94 of trucks parking during nighttime hours on mainlane shoulders and along entrance and exit ramps of rest areas. Recommendations from a task force included (a) stricter enforcement to keep trucks off shoulders and ramps, (b) 2-hr restrictions on the length of stay of trucks at rest areas, and (c) provision of information on appropriate overnight truck parking facilities at rest areas and through press releases (*11*).

As a result of 10 fatalities occurring over a 5-year period involving vehicles parked on shoulders, Columbus, Ohio, has reduced the time period allowed for any vehicle to be parked on the right shoulder of a freeway. Effective in November 1989, the time period that a vehicle could remain on the shoulder, away from an interchange, was reduced from 12 to 3 hr. Near an interchange or at specified "hazardous" locations, a vehicle is now cited and towed immediately.

## URBAN TRUCK INSPECTION STATIONS

Another strategy for reducing truck accidents is increased commercial vehicle roadside safety inspections within large urban areas. This sometimes requires the construction of urban inspection stations. Because the park-and-ride lot at the I-95/I-495 north interchange met with limited success, Maryland DOT converted the remaining unused portion into an inspection facility for trucks. Approximately 3,500 vehicles a year are inspected at the facility.

Virginia opened an urban inspection station on I-95 (Capital Beltway) at Van Dorn Street in October 1989. The construction cost of the Van Dorn Street inspection station in 1987 was $962,000. The estimated cost of building another inspection station on the Capital Beltway near I-66 is $3.5 million plus the cost of sound barriers. Reasons for the lower cost of the Van Dorn Street inspection station include available right-of-way and the use of an existing exit ramp.

Caltrans has an urban inspection station in Los Angeles on Interstate 405. This station, located on both sides of the urban freeway, was initially built as a weigh station. The northbound side was later modified to add six inspection bays on a paved asphalt surface. According to Caltrans officials, the estimated cost of building a complete truck inspection station with pits and building in urban areas is at least $8 million.

## INCIDENT MANAGEMENT

Publicly owned heavy-duty tow trucks and large cranes are used in Chicago. Illinois DOT maintains the heavy-duty tow truck fleet that currently patrols 100 centerline-mi of the Chicago freeways continuously. These "minutemen" respond to more than 100,000 incidents a year. Los Angeles maintains a traffic control team whose function is to reduce the number of secondary accidents by controlling traffic at the end of the queue caused by an incident. They respond to major incidents, which are defined as two or more lanes blocked for 2 hr or more. After each incident, a report is filed that includes the estimated delay to motorists and incident costs.

Tampa, Florida, has a contractual agreement with a private firm for the services of two heavy-duty wreckers on the Howard Frankland Bridge. These trucks are stationed at each end of the bridge and move to the opposite end every 30 min unless they are responding to an incident. Maryland, which is using a rotation list to contact private tow truck operators, has incident management teams in operation on all Interstate highways. Seattle is using an incident response van that houses four communication systems, an illumination system for nighttime incidents, a means of placing flares on the roadway for immediate traffic control, and means to seal or pump from ruptured truck fuel tanks.

## SUMMARY OF FINDINGS

Because of the significant delays in addition to the injuries and fatalities resulting from truck accidents and incidents on urban freeways, several operating agencies have investigated and implemented countermeasures to reduce truck accidents on urban freeways. Some of the countermeasures, such as increased enforcement, are designed primarily for trucks. Others apply to all traffic with specific elements for trucks, as when heavy-duty tow trucks are used to retrieve overturned trucks as part of an incident management program.

## FUTURE RESEARCH EFFORTS

The information in this paper comes from a literature review, telephone interviews with representatives of selected agencies, and site visits conducted in an FHWA project (*12,13*). In most cases, implemented truck accident countermeasures were not thoroughly evaluated by responsible agencies to determine their effectiveness. Frequently, agencies do not have the resources to conduct an analysis, or limited funding hinders agencies in evaluating the countermeasures.

Information on the actual rather than perceived effectiveness of the countermeasure, cost of the countermeasure, and transferability of the measure to different circumstances should be developed. Future research efforts should be channeled into analyzing promising countermeasures identified in this research such as incident response management, tall reinforced concrete barriers, and ramp improvements.

## ACKNOWLEDGMENTS

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*Abridgment*

# Traffic Engineering Evaluation of State Driver Licensing Manuals

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The contents of state driver licensing manuals were reviewed from a traffic engineering standpoint. On the basis of a review of published materials and discussions with educators and engineers, a list of topics was identified against which each manual was evaluated. By identifying the extent to which the manuals covered current traffic operational features, specific conclusions were reached and recommendations were made. The results indicated that driver licensing manuals do not adequately cover traffic operational topics. Each manual had strong points, but none covered every area examined in the research. Many manuals depicted important traffic control devices, but few actually gave satisfactory explanations of a device's meaning or location. A model manual that contains suggested topics is recommended as a first step in improving the manuals from a traffic engineering viewpoint. Better interaction and cooperation between the motor vehicle administrators and public relations personnel who write the manuals and engineers who implement traffic control features will ensure that the manuals reflect current highway practice.

Because of new approaches, techniques, and features associated with highway design, operation, and maintenance, motorists are encountering new and perhaps more sophisticated driving situations and traffic control devices. The driving public may not understand completely the meaning, purpose, or safe operation of these design features and traffic control devices. Examples include use of freeway acceleration and deceleration lanes, use of two-way left-turn lanes, layouts and traffic control of work zones, and meanings of symbolic signs. Thus, problems may develop that will lead to traffic accidents and inefficiencies in traffic flow if drivers are not educated further. Educating the driving public about the operation and meaning of these changes and new situations is a major challenge for engineers, educators, motor vehicle administrators, and enforcement agencies.

One important educational tool that all states use to inform and teach at least prospective licensed drivers is the driver licensing manuals published by state departments of motor vehicles. A driver licensing manual is typically a printed, pocket-size document that contains information about rules, laws, and regulations for driving in a particular state. The manual also explains highway terminology, defines the legal meaning of certain traffic control devices, and illustrates a variety of driving situations that may be encountered in rural and urban settings. State driver licensing manuals are the principal media for transferring the latest highway advances to the driving population of the future, so it appears appropriate to examine them to see how well the latest traffic control technology is incorporated and to identify deficient areas so that corrective action can be taken. The end result should be improved highway safety.

The overall goal of this work was to evaluate, from a traffic engineering standpoint, the contents of state driver licensing manuals. To meet this overall goal, several specific objectives were developed, which were

1. To conduct a comprehensive review of traffic operations standards and guidelines;
2. To identify, on the basis of the literature review and discussion with educators and practitioners, recently developed traffic control devices that may be causing problems for drivers because of lack of knowledge or experience with them;
3. To obtain and review carefully each state's driver licensing manual relative to its coverage of the topics identified in the previous objective; and
4. To make recommendations to improve the manuals from a traffic engineering standpoint.

## METHODOLOGY

A key task in the research was to identify recently developed or implemented traffic control devices and techniques that may be causing problems for drivers because they lack knowledge about or experience with a particular feature. This was accomplished through a review of pertinent literature and discussions with engineering educators and practitioners. Major references (textbooks, standards, and manuals) dealing with traffic engineering and highway safety were reviewed with an eye toward identifying recently developed features and devices and preparing a list of potentially misunderstood highway features.

The list was also based on formal and informal discussions with practicing engineers at professional society meetings and training courses. Another source of input was Eck's experience in accident reconstruction. Such in-depth analysis of accidents generates information, especially in the area of human factors, that is not typically available from accident records. The literature review did not locate any data relative to driving difficulties or accidents experienced by drivers that could be traced to lack of knowledge or to the fact that specific information was missing in the manuals. Because of the large number of interrelated factors present in most motor vehicle accidents, it is not possible to determine from accident statistics that lack of understanding of traffic control elements contributed to an accident. However, the list generated was thought

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to be reasonable because it was based on information from different sources.

For each of the traffic control elements identified, a set of questions was developed to be used for evaluating that element. To produce an objective, reproducible technique, each question required a "yes" or "no" response. The questions were developed so as to show how well each traffic engineering topic was addressed in the manuals. Although the specific criteria are too numerous to present here, an example is included for illustrative purposes.

### Example

#### *Topic*

A two-way left-turn lane is a continuous center lane in which left turns are permitted in both directions. Such lanes are typically found in places where an arterial passes through a developed area with many street and driveway intersections and where it is impractical to limit left turns. Geometrically, a two-way left-turn lane is located between opposite lines of traffic and provides a bay that harbors left-turning vehicles. This lane is sometimes misused as a passing or acceleration and deceleration lane. Drivers must be aware of the purpose of two-way left-turn lanes and be familiar with the signing and pavement markings associated with such lanes; they must understand how to use them.

#### *Questions*

Questions to be considered for this criterion ask if the manual discusses

- What a two-way left-turn lane is,
- Where such lanes can be found,
- A desirable procedure for negotiating such lanes, and
- Signing and pavement markings associated with such lanes.

### Evaluation

The first step in acquiring the licensing manuals for evaluation was to send a letter describing the nature of the research and requesting a copy of the current (early 1989) edition of the driver licensing manual to the department of motor vehicles for each of the 50 states, the District of Columbia, and Puerto Rico. Manuals were received from 49 states and Puerto Rico. It was subsequently learned that the District of Columbia does not use a traditional driver licensing manual.

Each of the manuals was carefully reviewed with an eye toward answering the questions just outlined. The researchers often had to use judgment in determining if the question was answered in the context of this research. A data collection form was developed to make tabulating the results more convenient.

## RESULTS

A summary of the percentage of manuals with affirmative responses to each of the traffic control device questions is presented in Table 1. This section highlights some of the specific findings.

As expected, assignment of specific colors and shapes to particular classes of signs was well covered in the manuals. Except for those of a few states, the manuals contained full-color representations of the different classes of signs. Use of color is thought to be critical in conveying the meaning of different classes of signs.

Most of the signs identified in this study were shown in the manuals. Several signs—for example, limited sight distance, added lane, divided highway, and chevrons—did not receive adequate coverage in the opinion of the researchers.

Although pictorial representations of the signs were shown and labeled, very few manuals discussed the meanings of the signs or the places they could be expected to be located. The researchers believe that the functions of the signs should be explained and that plan views should be included to show typical locations at which the signs are installed.

**TABLE 1 Summary of Affirmative Responses to Traffic Control Device Questions**

| Traffic Control Device | Number of Affirmative Responses (Total Responses = 50) | % of Manuals with Affirmative Responses |
|---|---|---|
| 1. Sign Color/Shape | | |
| Color | 46 | 92 |
| Shape | 47 | 94 |
| Full-color Graphics | 47 | 94 |
| 2. STOP/YIELD Ahead Sign | | |
| Graphic of Sign | 17 | 34 |
| Meaning and Purpose | 9 | 18 |
| 3. Turn/Curve Sign | | |
| Graphic of Signs | 34 | 68 |
| Difference Explained | 20 | 40 |
| 4. Signal Ahead Sign | | |
| Graphic of Sign | 43 | 86 |
| Meaning and Purpose | 19 | 38 |

(*continued on next page*)

**TABLE 1** *continued*

| Traffic Control Device | | Number of Affirmative Responses (Total Responses = 50) | % of Manuals with Affirmative Responses |
|---|---|---|---|
| 5. | Added Lane Sign | | |
| | Graphic of Sign | 2 | 4 |
| | Meaning and Purpose | 1 | 2 |
| 6. | Work Zones | | |
| | Signs and Markings | 44 | 88 |
| | Importance of Speed Limits | 0 | 0 |
| 7. | Lane Drops | | |
| | Drop Defined | 10 | 20 |
| | Safe Procedure | 11 | 22 |
| | Graphic of Sign | 39 | 78 |
| 8. | Divided Highway Sign | | |
| | Graphic of Sign | 5 | 10 |
| | Meaning and Purpose | 4 | 8 |
| 9. | Limited Sight Distance Sign | | |
| | Sight Distance Concept | 0 | 0 |
| | Problems | 1 | 2 |
| | Graphic of Sign | 0 | 0 |
| 10. | Chevron Alignment Sign | | |
| | Graphic of Sign | 6 | 12 |
| | Purpose and Intent | 5 | 10 |
| 11. | Narrow Bridge Sign | | |
| | Graphic of Sign | 18 | 36 |
| | Purpose and Intent | 7 | 14 |
| 12. | Hill Sign | | |
| | Graphic of Sign | 41 | 82 |
| | Meaning and Purpose | 20 | 40 |
| 13. | Truck Escape Ramp Signing | | |
| | Types of Signs | 0 | 0 |
| | Symbols and Colors | 0 | 0 |
| 14. | HOV Lane Signing | 7 | 14 |
| 15. | Traffic Signals | | |
| | Meaning of Colors | 49 | 98 |
| | Solid vs. Flashing | 48 | 96 |
| | Change Interval | 30 | 60 |
| 16. | Actuated Signals | | |
| | Definition | 1 | 2 |
| | Recognition | 0 | 0 |
| 17. | Left-turn Signals | | |
| | Graphics of Displays | 45 | 90 |
| | Meanings of Displays | 47 | 94 |
| | Associated Signing | 11 | 22 |
| 18. | Pavement Markings | | |
| | Meanings of Colors | 45 | 90 |
| | Solid vs. Broken | 49 | 98 |
| 19. | Two-Way Left-Turn Lanes | | |
| | Definition/Description | 28 | 56 |
| | Geometry of | 28 | 56 |
| | Procedure for Use | 16 | 32 |
| | Associated Signing | 18 | 36 |
| | Associated Pavement Markings | 27 | 54 |

In general, the manuals did a good job of discussing the meaning of the different color indications found on a traffic signal head. The difference between solid and flashing signals was also well covered. Laws and signing pertaining to right turns on red were usually explained in the section on traffic signals. However, traffic-actuated signals were not directly defined or described in any of the manuals.

Discussion of the different colors and the meanings of solid versus broken longitudinal pavement markings were judged adequate in most manuals. Full-color representations of the pavement markings were shown in most manuals.

## CONCLUSIONS AND RECOMMENDATIONS

Overall, the manuals appeared to do a fair job in showing traffic control devices but a poor job in explaining the meanings of the devices. Most manuals did not sufficiently describe the meanings and locations of traffic control devices. Many manuals showed a traffic control device included in the criteria but merely labeled it without detailing its function. Without a description of the meaning and location of a particular traffic control device, motorists may not know how to interpret and react to it.

Overall, it appeared that no set format was followed for the organization and layout of the manuals. No two manuals appeared to be alike. Although standardization is probably undesirable, there should be at least some minimum set of topics that all manuals should cover. Some uniformity and consistency would be helpful.

Similarly, there must be an arrangement of material that would be optimum from the viewpoint of information transfer. The manuals lacked a consistent approach in terms of the organization of individual sections dealing with particular topics. Many manuals related information on one subject over several, unconnected pages. It would be desirable to show all available information about one subject in one section as opposed to throughout the manual. In this way, the reader can better appreciate interrelationships.

There appears to be only limited coordination between traffic engineers and those who prepare driver licensing manuals. With highway technology changing almost daily, engineers should have a defined procedure for transferring new advances directly to motor vehicle administrators. Problems that arise because of the lack of coordination include manuals that show signs that are no longer in use and manuals that do not show newly developed signs.

If a state does not use a particular traffic control device on its highway system, discussion of that device typically will not be incorporated into its licensing manual, even though surrounding states may use that device. In a society that is as mobile as ours, drivers travel readily between states and it is important that they be able to recognize and interpret common traffic control devices that may not be used in their own states.

As stated, driver licensing manuals should not be standardized. Each state has unique areas that only it can address properly. However, all manuals probably should include a predetermined minimum amount of information about engineering-related subjects relative to highway safety. The following recommendations are intended to improve the manuals' coverage of traffic engineering topics:

- Engineers should play a bigger role in manual development. This includes engineers' working directly with the manual developers to determine jointly the best combination of engineering and educational topics that would enhance highway safety.
- Full-color representations of signs and pavement markings are a necessity to ensure that the correct expectancies can be developed about a sign or marking.
- Signs shown in the manuals should not be merely labeled but also discussed briefly as to their purpose and meaning. If possible, the signs should be shown in conjunction with plan views of highway situations (e.g., an intersection) so that the driver will know where to look for a particular sign.
- States should have a system or procedure for publicizing revisions to the manuals so that currently licensed drivers can keep up to date with changes.
- A model manual that contains the minimum amount of information necessary in a manual should be developed.

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# Evaluation of Variable Message Signs: Target Value, Legibility, and Viewing Comfort

JONATHAN UPCHURCH, JEFFREY D. ARMSTRONG, M. HADI BAAJ, AND GARY B. THOMAS

Three different technologies for variable message signs were evaluated in terms of target value, legibility distance, and viewing comfort. The technologies evaluated were flip disk, light-emitting diode (LED), and fiber optic. For comparison purposes, conventional overhead guide signs were also evaluated. Twelve signs were evaluated in the field in a human factors study; hired observers measured target value and legibility distance from a moving vehicle on the freeway and subjectively evaluated viewing comfort. Observations were made under four lighting conditions: midday, night, washout, and backlight. For target value, legibility distance, and viewing comfort, fiber-optic signs performed better than LED signs in most conditions. However, both types have acceptable performance overall. The effects of observer age were identified and documented. Both fiber-optic and LED signs are recommended as acceptable for the freeway management system in the Phoenix, Arizona, urban area.

Variable message signs are one way an intelligent vehicle-highway system can communicate real-time traffic and incident information to the driver. Variable message signs are seeing more widespread use, and their technology is changing rapidly. This paper presents results of a study evaluating three different sign technologies. The results will be useful to those who are planning and designing freeway management systems and other applications that use variable message sign technology.

Several technologies have been developed for variable message signs, including shuttered fiber optic, light-emitting diode (LED), electromagnetic flip disk, fiber-optic–enhanced flip disk, lamp matrix, and liquid cell. The first three technologies—shuttered fiber optic (referred to simply as "fiber optic"), LED, and flip disk—were evaluated in this study.

Four fiber-optic signs and two LED signs were installed on the Phoenix urban area freeway system in February 1991 for evaluation. Four flip-disk signs, which were already in use on the freeway system, were also evaluated. Two conventional overhead guide signs were included in the experiment for comparison purposes. Thus, 12 signs were evaluated in the study.

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## STUDY OBJECTIVE

To be effective, a variable message sign must

- Attract the motorist's attention;
- Be legible and provide sufficient legibility distance for the driver to read the sign at freeway speeds;
- Cause minimal visual discomfort to the driver; and
- Be effective under a variety of lighting conditions (e.g., bright daylight, night, and low sun angles).

The study evaluated each of the three variable message sign technologies for target value, legibility distance, and viewing comfort. This analysis revealed the relative performance of each technology and determined whether each technology performed at an acceptable level in each category.

## DESCRIPTION OF SIGNS

Each of the fiber-optic and LED signs had three rows of legend with 18 characters in each row. Each character is formed by an array of 35 pixels 7 high and 5 wide. The flip-disk signs also had three rows of letters. Fiber-optic characters are 16.1 in. high, LED characters are 17.8 in. high, and flip-disk characters are approximately 18 in. high. The LED sign face has a nonglare polycarbonate sheet.

The conventional signs (white letters on a green background) used in the evaluation were interchange sequence signs, with 13.3-in. capital letters and 10.0-in. lowercase letters. Both conventional signs used high-pressure sodium overhead lighting fixtures to illuminate them at night. All signs were mounted overhead. Table 1 gives the numerical designation, type of technology, location, type of structural support, and directional orientation of each sign used in the analysis.

## TARGET VALUE

Target value describes how noticeable a sign is or how well it attracts the motorist's attention. The effectiveness of a sign and the length of time the motorist has to read a sign may depend on target value.

TABLE 1 Sign Type, Location, and Description

| SIGN | TYPE(1) | LOCATION | STRUCTURE | DIRECTION(2) |
|---|---|---|---|---|
| VMS1 | F.O. | I-10 WB at RAY ROAD | OVERPASS | NB |
| VMS2 | F.O. | I-10 WB at GUADALUPE RD | OVERHEAD | NB |
| VMS3 | LED | SR360 WB at McCLINTOCK | OVERHEAD | WB |
| VMS4 | F.O. | I-17 SB at CENTRAL AVE | OVERHEAD | EB |
| VMS5 | FLIP | I-10 WB at 13th STREET | OVERHEAD | WB |
| VMS6 | FLIP | I-10 WB at DECK TUNNEL | TUNNEL | WB |
| VMS7 | FLIP | I-10 EB at 21st AVE | OVERHEAD | EB |
| VMS8 | FLIP | I-10 EB at DECK TUNNEL | TUNNEL | EB |
| VMS9 | LED | I-10 EB at 10th STREET | OVERHEAD | EB |
| VMS10 | F.O. | I-10 EB at 19th STREET | OVERHEAD | EB |
| CS1(3) | CONV | I-17 SB near 16th ST. | CANTILEVER | EB |
| CS2(4) | CONV | I-10 WB near 35th AVE. | OVERPASS | WB |

1 Key to Sign Type Definition
F.O. = Fiberoptic
LED = Light Emitting Diode
FLIP = Flip-Disk
CONV = Conventional

2 The directional orientation of the approach roadway at the sign location.

3 Conventional Sign 1 is an interchange sequence sign reading,

16th St 1/4

Sky Harbor Airport 1 1/4

4 Conventional Sign 2 is an interchange sequence sign reading,

35th Ave 1/4
43rd Ave 1 1/4
51st Ave 2 1/4

Previous studies of target value have been done in highly controlled laboratory settings. In this study a very simple measure of target value was used. As observers approached a sign, they were asked to look for it. The distance to the sign from the first point it was noticed is the target value.

## LEGIBILITY DISTANCE

Legibility distance is the distance from which a driver is able to read a sign. Sign legends must be large enough that the driver has enough time to read the message and respond safely. For overhead guide signs on freeways, standard alphabets, stroke widths, and letter sizes have been adopted to assure adequate legibility. For nighttime legibility, standard levels of illumination have also been adopted.

No standards have been set for variable message signs to guarantee adequate legibility. The formation of letters by a matrix of lamps, disks, or light sources is very different from a standard alphabet on a conventional sign. Therefore, it is expected that the legibility distance is quite different from the 50 ft of legibility distance for each inch of letter height provided by standard alphabets.

## VIEWING COMFORT

Viewing comfort describes any discomfort caused by glare or harshness of light. In a dark driving environment, a brightly lit sign is sometimes so bright that it causes discomfort for a driver because the eye is slow to adapt to a bright light source in dark surroundings. Similarly, after a driver passes the sign, the eye may adapt slowly to the dark surroundings, causing discomfort due to the inability to see well. These effects are more pronounced in locations where ambient light levels are very low (rural and semirural areas) and less pronounced where ambient light levels are medium to high (such as urban freeways). Older drivers are more sensitive to, and take longer to recover from, glare. The large illuminated area on a variable message sign may cause glare or discomfort problems. Some variable message signs have a light output adjustment feature that can reduce this potential problem.

## OBSERVER GROUP

Target value, legibility distance, and viewing comfort were evaluated by 62 hired observers. Each observer was seated as a passenger in a moving automobile, so that signs could be viewed under dynamic conditions.

Of the 62 observers, 31 were between the ages of 18 and 31 (18 men, 13 women; average age of 21), and 31 were between 60 and 79 (16 men, 15 women; average age of 66). Past research has confirmed the substantial effects of age on visual performance. In technical terms, these effects include a decrease in amplitude of accommodation, reduction in pupil size, decrease in rate and amount of dark and light adaptation, loss of transmission of light due to increased opacity of the eye media, reduction in sensitivity (especially at low luminance levels), and degenerative changes in the various parts of the visual system, including the retina. Within the highway and traffic engineering community there is increasing concern about whether our roads and traffic control devices are designed to meet the needs of older drivers. For this reason, much of the study was dedicated to the evaluation of signs by older drivers.

Apart from targeting these specific age groups, no effort was made to recruit drivers with any special characteristics. However, all observers were required to be licensed drivers and to pass a visual acuity test for 20/40 vision.

## STUDY DESIGN

Studies were conducted four times a day between February 25 and March 30, 1991. Signs were evaluated under four lighting conditions to determine the effects of the position of the sun with respect to the signs. Observations were taken immediately after sunrise, during midday, just before sunset, and at night. Nighttime observations began about an hour after sunset, and midday observations began at about 10:30 a.m. The early morning and late afternoon studies allowed for the analysis of the backlight and washout conditions on the eight variable message and two conventional signs on east-west roadways.

Backlight describes conditions in which the sun is directly behind the sign (in front of the driver). This condition causes a strong silhouette that makes the sign more difficult to read. Washout describes conditions in which the sun is directly behind the driver, just above the horizon, and shines directly on the face of the sign, causing a glare or reflection on the sign face.

Washout and backlight represent the most severe tests of legibility and viewing comfort. Washout and backlight effects depend on the position of the sun with respect to the sign and the observer. The observation dates used in this study (from February 25 to March 30) were just before and after the vernal equinox (March 21). At the equinox the sun rises directly in the east and sets directly in the west. For signs on east-west roadways, the observation dates happened to provide a more rigorous evaluation of washout and backlight than would have been possible at other times of the year (when the sun is rising and setting farther to either the north or the south).

One group of 31 observers observed signs in the early morning and late afternoon (washout and backlight conditions), and a second group of 31 observers observed signs during the daytime and nighttime. Thus, each of the 62 individuals observed each sign under two lighting conditions.

The study team recognized that repeated use of the same observer had some disadvantages. Target value measurement is less reliable for a repeat observer because the observer becomes familiar with the sign locations.

Test messages were placed on the signs to evaluate the legibility distance. The first line of the test message read "SYSTEM TEST," a message designed to maintain credibility with the public during the month-long test. The second line contained a randomly generated string of six letters, similar to an eye chart, with a space between each letter. Random letters were used to provide a more rigorous test of legibility.

Letter combinations were changed after the midday evaluations so that observers did not see any combination more than once during the day. Twenty test messages were rotated among the 10 variable message signs twice each day, to eliminate any bias caused by the possibility that some combinations may have been easier to read than others.

This study was conducted under field conditions, not in a controlled laboratory environment. It was possible to control many variables and confounding factors, but not all of them. Those that could not be controlled included weather conditions, such as rain, and the contrast ratios on the signs. Uncontrolled contrast ratios may have influenced the measurements of target value and legibility distance.

## NEEDED LEGIBILITY DISTANCE AND TARGET VALUE

An important part of the study determined the amount of legibility and target value that is necessary to provide adequate viewing of the signs.

### Legibility Distance

A variable message sign must be legible from some distance so that the driver, at a typical travel speed, has enough time to read the message. On the basis of a review of previous research, a minimum exposure time of 6.0 sec on a three-line sign is recommended for unfamiliar drivers (*1*). This recommendation is based on an 85th-percentile reading time. Assuming some vehicles travel 60 mph (88 ft/sec), 6.0 sec is equal to 528 ft.

As a driver approaches an overhead sign, sign readability becomes restricted by the vertical cutoff angle of the windshield. This means that the sign will become hidden from the motorists's view as the vehicle nears the sign.

Messer and McNees discuss this aspect and cite other publications that recommend a vertical cutoff angle of 7.5 degrees (*2*). For the existing variable message signs, the average height to the center of the sign is 23.0 ft. Average driver eye height can be assumed to be 3.5 ft. The vertical displacement of 19.5 ft combined with the 7.5-degree cutoff angle means that the sign becomes obscured by the vehicle roof at a distance of 150 ft.

Combining a minimum of 528 ft for reading distance with 150 ft because of vehicle cutoff results in a distance of 678 ft. To be acceptable, a sign with a three-line message should be legible from a distance of no less than 678 ft.

### Target Value

Although there is no commonly accepted rule of thumb for how much target value is needed, it is generally agreed that more is better. For this project a conservative value of two times the legibility distance was selected. This means that once a driver notices a sign, he or she can devote time to other driving tasks before devoting time to reading the sign. Thus, 1,356 ft is the desirable (acceptable) target value for variable message signs.

### Viewing Comfort

The researchers selected a discomfort rating less than or equal to 1.0 as acceptable (equivalent to little discomfort).

## STUDY PROCEDURE

A 20-min orientation preceded the field test. The purpose and importance of the study were emphasized. The observers were taught the meanings of target value, legibility distance, and viewing comfort and were asked to explain each of these factors to the experimenter to demonstrate that they had a satisfactory understanding of the meanings. They were shown photographs of various types of freeway signs, including variable message signs, so that they would know what to look for on the freeway.

A visual acuity test was given to each observer, in which they were to read three lines of an eye chart from a distance of 20 ft to test for 20/30, 20/20, and 20/15 vision. If these tests were failed, they were tested for 20/40 vision, which is the minimum allowable visual acuity level to qualify for a driver's license. Observers were instructed to wear contact lenses or glasses if so required by their driver's licenses.

Of the 31 observers in the younger group, 16 wore glasses and 5 wore contact lenses. Of the 31 observers in the older group, 28 wore glasses and none wore contact lenses. The average corrected vision of the younger observers was 20/17 (21 observers with 20/15 vision, 9 observers with 20/20, and 1 observer with 20/30). The average corrected vision of the older observers was 20/22 (14 observers with 20/15 vision, 8 observers with 20/20, 4 observers with 20/30, and 5 observers with 20/40).

Once the observers demonstrated an understanding of the parameters involved in the project, the field test began. The test involved a 67-mi drive on the Phoenix freeway system. Observers were seated in the front passenger seat of an automobile. One experimenter drove the vehicle, and a second was seated in the back seat to record data. If safety permitted, the vehicle was driven at a constant speed of 55 mph.

Two practice signs (conventional overhead guide signs) were analyzed by each observer before testing to ensure that they understood the parameters that they were to evaluate and the equipment used to measure distances.

Target value and legibility distances were measured with a distance measuring instrument (DMI) made by Nu-metrics Instrumentation, Inc. The DMI was wired to a transmission sensor. After installation, the DMI was calibrated on a 1,000-ft test section of road.

The DMI was actuated when the observer first noticed a sign, when the observer could read the legend on the sign, and when the vehicle passed under the sign. Values for target value and legibility distance were calculated from the measured distances.

### Target Value

At distances of 1 to 2 mi before reaching each sign, observers were asked to start looking for a particular sign. The experimenters' instructions were similar to the following examples: "Begin looking for the next variable message sign"; "Evaluate the first green overhead sign beyond the next overpass." Observers actuated the DMI when they first saw the sign.

### Legibility Distance

Observers were asked to actuate the DMI when they were sure they could distinguish all six letters in the test message and to read the message aloud to the experimenter immediately to verify that they could read the message.

### Viewing Comfort

Observers were then asked to concentrate on the viewing comfort of each sign. During the orientation observers were informed that discomfort might be caused by any of the following factors:

1. Reflection of sunlight off of the sign (glare),
2. Reflection of headlights off of the sign (glare),
3. A sign that is too bright in comparison with its surroundings, and
4. The position of the sun behind the sign.

After passing the sign, they were asked to rate the discomfort of the sign as no discomfort, little discomfort, moderate discomfort, or high discomfort. Observers were asked to determine for themselves what each level of discomfort meant but to use a consistent scale in rating the 12 signs.

## FINDINGS AND ANALYSIS

Table 2 presents a detailed tabulation of target value, legibility distance, and viewing comfort; findings related to this table are described in the following paragraphs. Table 2 uses bold type to indicate technologies with acceptable target value, legibility distance, and viewing comfort. Use of the word "significant" in the following paragraphs means statistically significant with 95 percent confidence.

### Target Value

The analysis of the target value of each technology was difficult because potential target values for some signs were constrained by roadway geometry. In some cases, the location at which an observer noticed the sign was influenced by horizontal or vertical curvature or overpasses obstructing the line of sight as much as it was influenced by the sign's attracting the observer's attention. The maximum possible target value (longest possible line of sight) for each sign was measured by the experimenters; it ranged from 1,567 to 9,221 ft.

To compare adequately the target values of the different technologies, four signs (one of each type) were selected for

**TABLE 2 Comparison of Target Value, Legibility Distance, and Viewing Comfort for Each Technology**

| | | FIBEROPTIC | LED | FLIP-DISK | CONVENTIONAL |
|---|---|---|---|---|---|
| TARGET VALUE (feet) | | | | | |
| Maximum Possible | | 3286 | 2886 | 2811 | 3200 |
| Observed Values | | | | | |
| Mid-day | Y(1) | **3087** | **2634** | **2544** | **2229** |
| | O(2) | **2841** | **2499** | **2591** | **1713** |
| | A(3) | **2960** | **2562** | **2568** | **1962** |
| Night | Y | **2958** | **2514** | 884 | **2078** |
| | O | **2701** | **2004** | 898 | **1600** |
| | A | **2830** | **2249** | 891 | **1839** |
| Backlight | Y | **2467** * | **1659** * | **1657** | 1285 * |
| | O | 1080 * | 1170 ** | 1270 | 928 * |
| | A | **1873** | **1433** | **1442** | 1097 |
| Washout | Y | **2994** * | **2331** * | **2317** | **2003** * |
| | O | **2350** * | **1950** * | 1067 | **1436** * |
| | A | **2708** | **2162** | **1692** | **1736** |
| LEGIBILITY DISTANCE (feet) | | | | | |
| Mid-day | Y | **1006** | **812** | **731** | |
| | O | **959** | **681** | 667 | |
| | A | **983** | **743** | **698** | |
| Night | Y | **687** | **794** | 363 | |
| | O | 667 | 602 | 348 | |
| | A | **678** | **694** | 355 | |
| Backlight | Y | **782** * | 616 * | 263 | |
| | O | 535 * | 337 ** | 177 | |
| | A | 659 | 502 | 219 | |
| Washout | Y | **882** * | 554 * | 472 | |
| | O | **817** * | 400 * | 363 | |
| | A | **853** | 487 | 420 | |
| VIEWING COMFORT (discomfort rating, 3 = high discomfort) | | | | | |
| Mid-day | Y | **0.5** | **0.6** | **0.9** | **0.1** |
| | O | **0.5** | 1.3 | 1.8 | **0.7** |
| | A | **0.5** | **1.0** | 1.4 | **0.4** |
| Night | Y | **0.6** | **0.4** | 2.2 | **0.1** |
| | O | **0.5** | **0.8** | 2.6 | **0.4** |
| | A | **0.5** | **0.6** | 2.4 | **0.3** |
| Backlight | Y | 1.7 * | 1.8 * | 2.5 | 1.7 * |
| | O | 2.0 * | 1.6 ** | 2.6 | 1.7 * |
| | A | 1.9 | 1.7 | 2.5 | 1.7 |
| Washout | Y | **0.5** * | 1.8 * | 2.0 | **0.3** * |
| | O | **0.5** * | 1.7 * | 1.9 | **0.6** * |
| | A | **0.5** | 1.8 | 1.9 | **0.5** |

(1) Y = Younger observers
(2) O = Older observers
(3) A = All observers
* Sample size < 26
** Sample size < 13

The values shown are the mean in each category.

Acceptable values are shown in bold type. Unacceptable values are shown in light type.

comparison. The four signs had similar maximum possible target values, as follows.

| *Sign* | *Type* | *Maximum Possible Target Value (ft)* |
|---|---|---|
| VMS5 | Flip disk | 2,811 |
| VMS9 | LED | 2,886 |
| CS2 | Conventional | 3,200 |
| VMS10 | Fiber optic | 3,286 |

Observations from these four signs were used to compare the target values of the four technologies. The mean target values for each of these four signs (as noted by observers) are compared to each other and to the desirable target value of 1,356 ft. Target values are compared for the four lighting conditions and for both age groups. All data are shown in Table 2. Care must be used in interpreting the data in Table 2 and in the following paragraphs. It is emphasized that there is some difference in the maximum possible target value of the four signs.

### *Midday Target Values*

All midday observations, except for those taken while it was raining, were considered in this analysis. For the fiber-optic, LED, and flip-disk signs, there are small differences in the mean target values as a function of either age or technology. The mean target value for the conventional sign is substantially lower.

All four types of sign had target values that exceeded the desirable target value of 1,356 ft. Target values for fiber-optic and LED signs ranged from 2,499 ft (LED, older observers) to 3,087 ft (fiber optic, younger observers).

### *Nighttime Target Values*

The analysis of nighttime target values includes all nighttime observations except for those taken while it was raining. The fiber-optic sign had the highest mean nighttime target value: 2,830 ft. The LED sign had a mean nighttime target value of 2,249 ft; it showed a larger difference between older and younger observers. The mean target value for the conventional sign was 1,839 ft.

The most striking difference, however, was with the flip-disk sign, which had a mean nighttime target value of only 891 ft.

The fiber-optic, LED, and conventional signs all had target values that exceeded the desirable target value of 1,356 ft. Flip-disk signs fell short; 898 ft for older observers and 884 ft for younger observers. The target values for fiber-optic and LED signs exceeded 2,000 ft.

### *Backlight Target Values*

This analysis includes all observations taken while the sun was shining behind the sign. Some of the backlight analysis is based on morning observations and some is based on afternoon observations, depending on the orientation of the signs.

The fiber-optic sign had the highest average target value: 1,873 ft. The LED and the flip-disk signs had similar target values of 1,433 and 1,442 ft, respectively. The mean target value for the conventional sign was 1,097 ft.

For the backlight condition, there is a greater difference in target values for older and younger observers than was noted during the daytime and nighttime observations. This difference is most notable with the fiber-optic signs, for which the mean target values were 2,467 ft for the younger observers and 1,080 ft for the older observers (who often commented about discomfort from sun in their eyes).

None of the four types of signs met the desirable target value of 1,356 ft for older observers. The conventional sign also fell short for younger observers. The other three types of sign exceeded 1,356 ft for younger observers.

### *Washout Target Values*

The washout analysis includes all observations taken while the sun was shining in front of the sign.

The fiber-optic sign had the highest target value for this condition, with an average of 2,708 ft. LED signs averaged 2,162 ft and flip-disk signs 1,692 ft.

There is a substantial difference in target values between the older and younger observers. The difference is most notable for the flip-disk signs, for which the mean target values were 2,317 ft for the younger observers and only 1,067 ft for the older observers. The older observers often commented that there was too much glare off of the flip-disk signs, making them difficult to recognize.

Except for the flip-disk sign when observed by the older group, all signs exceeded the desirable target value of 1,356 ft. The target values for fiber-optic and LED signs exceeded 1,950 ft.

## **Legibility Distance**

### *All Observations*

Average legibility distances for all variable message signs were as follows:

- Younger observers—687 ft
- Older observers—579 ft
- All observers—634 ft

The legibility distance of the variable message and conventional signs was not compared.

Mean legibility distance for each sign is illustrated in Figure 1. Figure 2 shows mean legibility distance for each technology on the basis of all observations of the signs (the numbers above the bars are sample sizes).

The fiber-optic signs had the greatest overall legibility distance throughout the study, followed by the LED and flip-disk signs. The overall legibility distance for the flip-disk signs was significantly lower than it was for the other three technologies; it was inadequate according to the 678-ft acceptable minimum.

The younger observers consistently had higher legibility distances than the older observers. Whereas the difference between age groups was relatively small for the fiber-optic

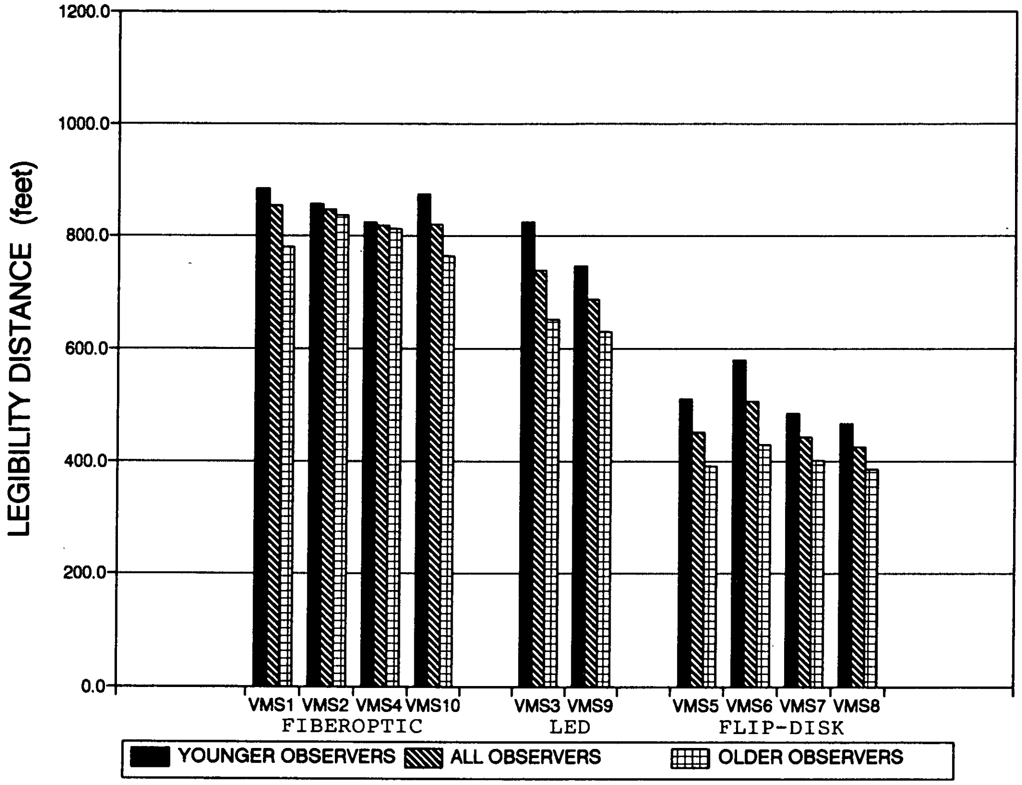


**FIGURE 1** **Mean legibility distance by sign, all observations.**

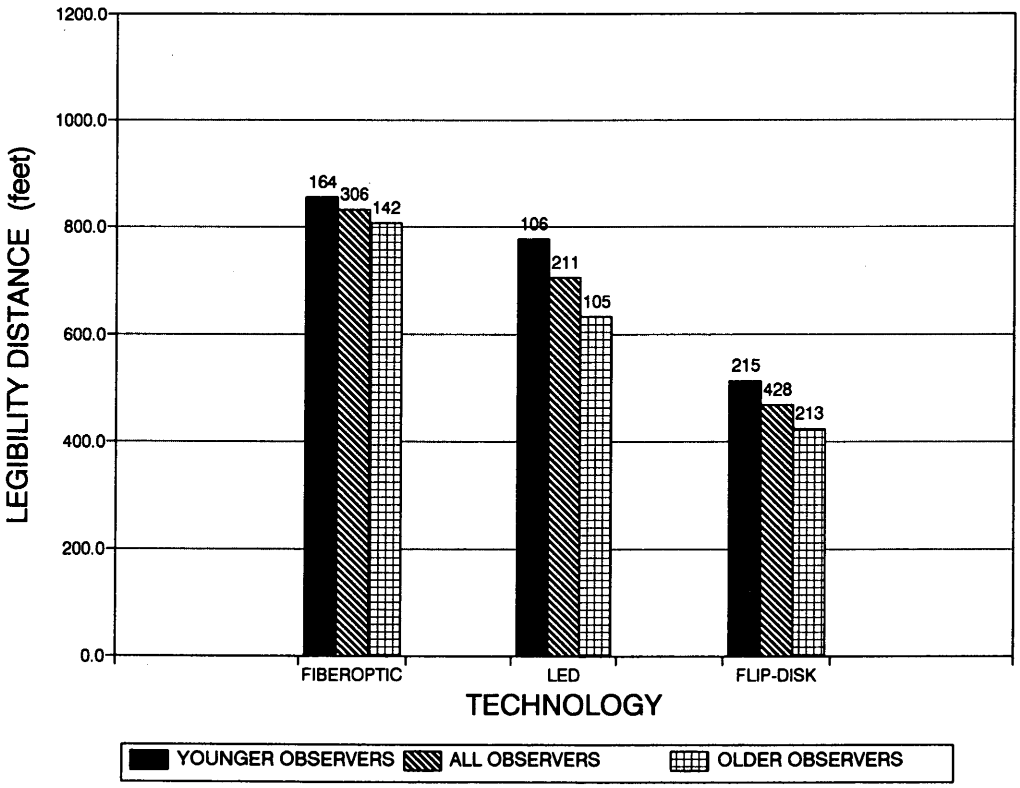


**FIGURE 2** **Mean legibility distance by technology, all observations.**

signs, Figure 2 shows that for the LED and flip-disk signs, legibility distance was approximately 100 to 175 ft better for the younger observers.

Figure 2 shows the sample mean for each technology. To demonstrate significant differences in legibility distances, a 95 percent confidence interval (lower and upper limits) can be used. This means that there is a 95 percent probability that the population mean (all drivers with the same characteristics as the observers) falls within the given interval above or below the sample mean.

The lower 95 percent confidence interval for the mean legibility distance of the fiber-optic signs was 798 ft, and the upper 95 percent confidence interval for the mean legibility distance of the LED signs was 750 ft. Thus, legibility distance for the fiber-optic signs was significantly higher than for LED signs. The flip-disk signs had by far the poorest legibility distance.

### *Midday Legibility Distance*

Figure 3 illustrates the average midday legibility distances for the three variable message sign technologies and for both age groups (the numbers above the bars are sample sizes). In the daytime, older observers had slightly lower legibility distances than the younger observers. This difference was largest with the LED signs.

Figure 3 shows the mean daytime legibility distances for each technology. On the basis of confidence intervals, the fiber-optic technology had a significantly higher legibility distance than the LED or flip-disk signs. All sign types met the acceptable legibility distance of 678 ft except for flip-disk signs for older observers.

### *Nighttime Legibility Distance*

The nighttime analysis includes all observations taken at night, except for those taken while it was raining. The LED signs had the highest overall nighttime legibility distance of the variable message signs. The average distance was just slightly higher than it was for the fiber-optic signs. The older observers had greater legibility distances for the fiber-optic signs than for the LED signs; the younger observers had greater legibility distances for the LED signs than for the fiber-optic signs.

Both observer groups had a low legibility distance for the flip-disk signs. There was little difference in legibility distances among age groups for the fiber-optic and the flip-disk signs, but there was a substantial difference among age groups for the LED signs—probably because the older observers more often experienced glare problems with the LED signs at night but never mentioned glare problems with the fiber-optic signs.

The fiber-optic and conventional signs were not significantly different at night, but the flip-disk had much poorer legibility at night.

The flip-disk signs fell far short of the 678-ft acceptable legibility distance. LED signs and fiber-optic signs for older drivers also fell short.

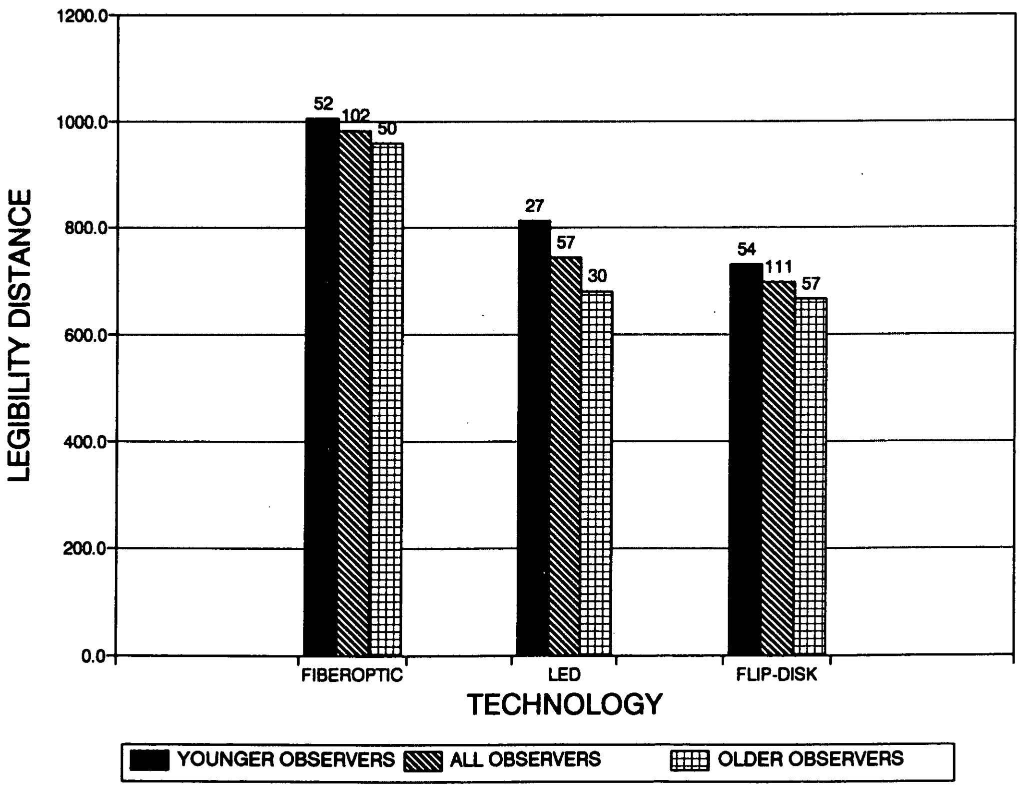


**FIGURE 3 Mean legibility distance by technology, daytime observations.**

### *Backlight Legibility Distance*

The flip-disk signs had significantly lower legibility distances in backlight conditions than the other two sign types. There was no statistically significant difference between the fiber-optic and LED technologies.

All three sign types failed to provide an acceptable legibility distance of 678 ft, although the fiber-optic signs exceeded this level for the younger observers.

### *Washout Legibility Distance*

Fiber-optic signs had the highest legibility distance under washout, followed by LED and flip-disk signs.

Legibility distances for the older observers were lower than they were for the younger observers. Older observers appeared to have a greater problem with the reflection of the sun off of the sign than the younger observers did. The legibility distance of the fiber-optic signs was probably superior to the LED and flip-disk signs under this condition because very little glare reflected off of the fiber-optic signs; observers often mentioned glare problems associated with the LED and flip-disk signs.

Of the three technologies, the fiber-optic signs performed significantly better than the LED and flip-disk signs. Fiber-optic signs (853 ft) exceeded the acceptable legibility distance. LED (487 ft) and flip-disk signs (420 ft) did not.

### *Legibility Distance in Rain*

Rain occurred during only 11 out of 124 observation studies (8 daytime studies and 3 nighttime studies). In most cases, legibility distance was less under rainy conditions.

## **Viewing Comfort**

For analysis, discomfort ratings were converted to a numerical scale (no discomfort = 0; little discomfort = 1; moderate discomfort = 2; high discomfort = 3).

### *Midday Discomfort Rating*

Table 2 shows that the conventional signs had the lowest discomfort rating. The fiber-optic signs had the lowest discomfort rating of all of the variable message signs, followed by the LED and flip-disk signs. An analysis of observers' comments indicates that the flip-disk signs were uncomfortable because the letters were not bright enough. Observers often mentioned that the letters were too dim and did not stand out against the sign background. The only repetitive comment about the discomfort of the LED signs was that observers thought that the letters were not bright enough on sunny days.

There was little or no difference in discomfort rating for the older and younger observers for the fiber-optic signs, but there was a large difference between the two age groups for the LED and flip-disk signs.

### *Nighttime Discomfort Rating*

Table 2 reveals very low discomfort ratings for the fiber-optic, LED, and conventional signs and very high discomfort ratings for the flip-disk signs. The only major difference between age groups was that the discomfort rating for LED signs was about twice as high for the older observers as it was for the younger observers. The older observers often stated that there was too much glare off of the LED signs.

Nearly all of the nighttime observers had difficulty with the flip-disk signs and associated a great deal of discomfort with them.

### *Backlight Discomfort Rating*

Table 2 reveals that the discomfort ratings for the fiber-optic, LED, and conventional signs were quite similar in backlight conditions with very little difference for the two age groups. The discomfort rating for the flip-disk signs was much higher than for the other signs.

With the sun shining directly in their eyes, observers often experienced great difficulty in reading the signs. Thus, discomfort ratings for the backlight condition are much higher than for the other lighting conditions.

### *Washout Discomfort Rating*

The fiber-optic and conventional signs had low discomfort ratings under washout conditions, whereas the LED and flip-disk signs had high discomfort ratings. There was little difference in the discomfort ratings of the two age groups.

The most common contributor to the discomfort during washout was the reflection of the sun off of the sign and into a driver's eyes. This was most prevalent with the LED and flip-disk signs because the sun tended to reflect off of the transparent cover on the front of the sign. The fiber-optic sign produced little or no glare: observers never mentioned glare as a problem in viewing the fiber-optic signs.

# **CONCLUSIONS**

From the findings and analysis, the following conclusions are made.

## **Target Value**

1. During the daytime, there are small differences in target values between the three types of variable message signs. All three technologies have acceptable performance.
2. During the daytime, all three types of variable message signs have higher target values than conventional freeway guide signs.
3. At night, fiber-optic and LED signs have higher target values than either flip-disk or conventional signs.
4. For all four lighting conditions, fiber-optic and LED signs have higher target values than conventional freeway guide signs.

5. The flip-disk signs have very poor nighttime target values.
6. In comparison to daytime conditions, target values decrease for nighttime, backlight, and washout conditions.
7. Younger drivers generally have higher target values than older drivers. In some cases there is a very large difference between younger and older drivers.
8. Bright sunlight and glare have larger negative effects on older drivers than on younger drivers.

**Legibility Distance**

1. Fiber-optic signs have significantly higher average legibility distances than the LED or flip-disk signs during midday and washout conditions.
2. During backlight conditions, the fiber-optic and LED signs have significantly higher average legibility distances than flip-disk signs.
3. At night, fiber-optic and LED signs have similar legibility distances. However, flip-disk signs have significantly lower legibility distances at night.
4. A comparison of legibility distances for variable message sign technologies versus an acceptable legibility distance of 678 ft shows the following:
   - Flip-disk signs provide acceptable legibility distance only in the daytime. Therefore, they are unacceptable.
   - All three sign types are deficient for the backlight condition.
   - Fiber-optic signs provide acceptable legibility distance overall.
   - Fiber-optic signs perform slightly better than LED signs overall.
5. Older observers generally have lower legibility distances than younger drivers.

**Viewing Comfort**

1. Flip-disk signs have the highest discomfort rating of all four sign types analyzed; this was consistently true for all four lighting conditions. Flip-disk signs have an unacceptable discomfort rating.
2. Glare and strong sunlight are major contributors to viewing discomfort.
3. Fiber-optic signs have the lowest discomfort rating of all three types of variable message sign during the midday and washout conditions.
4. For nighttime and backlight conditions, fiber-optic signs are about equal to LED signs; both are better than flip-disk signs.
5. Fiber-optic and LED signs have an overall acceptable discomfort rating.

Because this study was conducted under field conditions and not in a controlled laboratory environment, uncontrolled contrast ratios may have influenced the measurements of target value and legibility distance reported in this section.

The observer studies were successful in analyzing the various human factors associated with the use of the signs. Both fiber-optic and LED signs compare reasonably well with conventional signs. Flip-disk signs do not perform as well; they have unacceptable performance in most categories.

In terms of absolute performance, the fiber-optic technology performed better than the LED technology for target value, legibility distance, and viewing comfort. Both technologies had acceptable performance overall and in most categories. On this basis it is recommended that both fiber-optic and LED signs are acceptable for the freeway management system in the Phoenix, Arizona, urban area.

## ACKNOWLEDGMENTS

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# Evaluation of High-Speed Isolated Signalized Intersections in California

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High-speed isolated signalized intersections (HSISIs) are generally encountered after long uninterrupted and uncongested flow conditions, therefore catching some motorists by surprise. For this reason alone, approaches to HSISIs need to have safe and effective warning, control, and intersection geometric treatments. Characteristics at California HSISIs that relate to accident rates are identified. A new safety indicator was introduced: intersection approach accident rate. Variables investigated included advance warning signs with and without flashing beacons, signal timing and phasing, channelization, signal equipment configurations, shoulder widths and types, median widths and types, and approach speeds. Forty HSISIs out of the approximately 100 statewide were chosen for the analysis. Twenty were selected from the highest accident group and 20, from the lowest accident group. Statistical analysis identified relationships between approach variables and approach accident rates. The primary variables found to be significantly correlated to low accident rates on approaches to HSISIs were the presence of a separate left-turn phase, a raised median, wide paved shoulders, and an advance warning sign with a flashing beacon. A demonstration project and interim procedures for California are encouraged.

The primary purpose of the U.S. highway transportation system is the safe, fast, and convenient movement of vehicular traffic while contributing to society's overall quality of life. One part of that system is, of course, high-speed highway facilities that move vehicles rapidly with minimum interruption, one of the characteristics of many rural and some suburban state highways. Occasionally, it is necessary to install traffic signals on these roadways. Because these high-speed isolated signalized intersections (HSISIs) are often encountered after lengthy uninterrupted flow conditions, drivers are often surprised. Sometimes drivers do not expect the signals, do not pay close attention to the traffic control, or become 'hypnotized' by a long tangent segment. When signalized intersections are encountered under these conditions, there is concern about a higher potential for accidents, especially severe accidents. This situation makes it extremely important to make the high-speed approaches to intersections and the assignment of right-of-way at those intersections safe through effective warning and control measures. FHWA and the California Department of Transportation (Caltrans) recently sponsored research by Washington et al. in California that addressed this subject (*1*). This paper focuses on that research effort.

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## RESEARCH OBJECTIVES

This research paper identifies approach characteristics that affect accident rates at HSISIs on California state highways. The results of this research can be used to become better informed as to HSISI safety characteristics and features. Control measures for improving intersection safety can also be identified and, where appropriate, implemented. This knowledge may promote more uniform design and operation of HSISIs. There are two specific research objectives; namely, to identify control measures or actions that will most likely reduce accidents at HSISIs and to develop procedures for using the results of this research.

## SCOPE AND LIMITATIONS

The general scope of the research was to identify features or control measures that affect safety at HSISIs in California. By identifying these features, recommendations to improve approaches with high accident rates could be made. With recommended improvements, intersections could experience a decrease in accident rates.

Forty intersections were chosen for analysis. The intersections were to be representative of the most safe and the least safe HSISIs throughout the state of California. Accident data would be taken from the Caltrans-maintained Traffic Accident Surveillance and Analysis System (TASAS) data base. Although this research work was started in June 1989 and completed 2 years later, the accident data spanned from 1980 to 1990. Other data included advance warning, intersection geometry, and signal operations.

To identify variables for study, the research did not include review of any legal cases associated with HSISIs, because legal cases were few and, if pending, were sensitive to outside scrutiny. The research also did not attempt to review collision diagrams at selected intersections. Intersections were broken down into accident types sufficiently by the TASAS data base; therefore, collision diagrams would not have benefited the analysis.

The research involved only signalized intersections on California state highways. It was determined early in the study that the goal was to identify characteristics of signalized intersections that affect safety; therefore, presignalization data would not provide substantial useful information. The focus was to identify characteristics and features of HSISIs conducive to safe operation.

## LITERATURE REVIEW

Because HSISIs are often encountered by surprise, the approaching motorist needs effective advance warning, efficient traffic control, and safe geometric features. For these reasons, HSISIs need to be studied to determine which features contribute to the safety of these intersections. No other studies found by the authors examined California HSISIs as comprehensively as this study. Other studies considered particular aspects of HSISIs in greater detail, and these will be reviewed in this section. A comparison of the findings discovered in the literature review with the results of this analysis has been made later in this paper. The comments that follow give a summary of previous research procedures and findings.

A 1984 study identified a prioritized list of problems at rural intersections (*2*). The most pressing problems in decreasing order of importance were (a) rural expressways where signals were unexpected, (b) intersections hidden by horizontal curves, (c) rural expressways with heavy truck traffic, and (d) intersections hidden by crest vertical curves. Some other circumstances listed as problems were (a) speed, (b) left-turn traffic, (c) left-turning drivers' misjudgment of speed of oncoming traffic, and (d) high volume of left-turning traffic without left-turn phasing. Also listed were significant sun interference, advertising signs drawing attention away from signals, urban intersections operating near capacity, last car passage at intersection, and long-range visibility. Countermeasures used to reduce problems at these sites in decreasing order of popularity were (a) detector placement and amber time adjustment, (b) activated Red Signal Ahead sign, (c) Prepare To Stop When Flashing signs, and (d) the flashing Signal Ahead sign. The basis for countermeasure installations were rear-end and right-angle accidents, red violations, speed problems, and truck accidents.

Barnack and the New York Department of Transportation's Safety Operations Unit conducted a study to determine the current state of the art in detector placement at traffic-actuated isolated intersections (*3*). At such locations an option or dilemma zone, defined by locations depicting deceleration rates between 12 and 8 ft/sec, catch drivers in the indecision of the amber light indication. The four-lane highway at Route 32 at 335, Albany, New York, was a three-phase, fully actuated controller. The study concluded that rear-end and right-angle accidents increase as detector distance from the intersection decreases.

Agent studied 65 rural high-speed intersections in Kentucky (*4*). Forty-seven intersections were signalized, and 18 were stop sign–controlled. Approach grade and curvature were typically level and straight. Approach and intersection features such as phasing, lighting, signing, geometry, signal timing, and speed limits were all tabulated and summarized in tables. Important characteristics and features for a safe intersection were (a) adequate sight distance, proper change intervals, and highly visible signal heads; (b) a red clearance interval for both through phases of traffic; (c) a green extension system for the major roadway to consider left-turn phasing; and (d) an advance warning sign used at less safe intersections.

A 1975 study conducted by the Stanford Research Institute (SRI) gathered data from 558 intersections coupled with accident reports on the 4,372 accidents that occurred at those locations (*5*). The intersections studied were taken from three San Francisco Bay Area counties: Alameda, San Mateo, and Santa Clara. The study covered the period between June 1973 and June 1975. SRI recommended the following countermeasures for the accident-related features at intersections in decreasing order of significance:

1. Sight distance must be unobscured on all approaches to an intersection [higher average daily traffic (ADT) requires greater sight distance].
2. For street signs, black lettering on a white background is more effective than reflective lettering on a dark green background, which is described and recommended in the *Manual on Uniform Traffic Control Devices* (*6*).
3. Intersections with left-turn lanes and ADT between 10,000 and 20,000 tend to have higher accident rates than equivalent intersections without left-turn lanes.
4. Intersections lacking raised pavement markers along the centerline striping are less safe than those with them.

A 1976 study attempted to develop guidelines for improving intersection geometrics and safety at rural municipalities (*7*). Data from more than 300 intersections in 42 towns in Virginia were included in the study. Accident data at these locations were taken for 24 months and included 2,300 accidents. The study concluded that poor driver sight distance on any of the approaches to an intersection tends to correlate with a higher-than-normal angle accident rate and that the standardization of signal displays should reduce accidents at high accident locations.

A 1980 study by Van Maren examined 61 rural multilane intersections in Indiana (*8*). Geometric and accident data were taken from each site and studied from 1974 through 1976. Multilane intersections in Indiana were a serious safety problem, having 25 times as many accidents per intersection as the average rural intersection. These intersections also had 5 percent of the accidents but accounted for only 1 percent of the rural intersections. The study concluded the following about multilane rural signalized intersections with high accident rates.

1. The presence of stop-line pavement markings on both the major and minor roadways decreases the accident rate.
2. The right-angle accident rate was reduced by route markers or advance warning signs, the route markers being the more effective of the two.
3. The presence of a horizontal curve on the roadway and a skew of the two roadways increase the accident rate.

A fairly recent study examined the accident rates related to traffic signal clearance intervals at high-speed (45 mph or greater) signalized intersections throughout the United States (*9*). Regardless of how the accident rates were calculated, it was determined that intersection groups with clearance intervals requiring a deceleration rate of greater than 10 ft/sec to stop in time for red had higher average accident rates than did intersections with longer clearance intervals.

A research investigation lead by Hammer studied the effectiveness of traffic signals in reducing accidents (*10*). Ninety of the California intersections were modified, and 202 intersections were new. A before-and-after study method was used.

Findings and recommendations due to the study included the following:

1. Multiphase signal operations should be provided as well as separate storage slots for high-volume left-turn movements.
2. Twelve-inch lenses should be used for mast arm–mounted installations.
3. Signalized intersections with a base accident rate of less than or equal to 0.6 accidents per million vehicle-mi will not experience a decrease in the accident rates due to improvements.
4. When left-turn channelization is signalized at a three-leg intersection, left-turn channelization should also be provided on the main line.

The main objective of a 1985 study was to review current traffic engineering practice relative to accident countermeasures at high-speed signalized intersections (*11*). Through its literature review and questionnaire the study found the following:

1. There were high accident rates at hidden intersections or rural expressways where intersections are unexpected.
2. At such intersections rear-end accidents were the most pressing problem; right-angle accidents and red violations were also of concern.

The study determined that the most dynamic traffic-actuated devices are the flashing Red Signal Ahead sign and its variations, the Prepare To Stop When Flashing sign, and flashing strobe lights.

A study by Lyles evaluated the effectiveness of advance warning signs at unsafe or hazardous intersections (*12*). The study concluded that some sign messages and configurations have more recognition and generate more motorist recall than others.

## DATA FILES

To analyze intersection characteristics with regard to safety at HSISIs, a sample representative of all types and configurations of California HSISIs was needed. A list of criteria was established to ensure that the final data would be representative of HSISIs in California and be suitable for statistical analysis. The criteria used to develop the preliminary list of HSISIs were the following:

1. The intersection should be in a rural location;
2. The intersection must contain at least one approach with a posted speed of 50 mph or greater;
3. At least one of the approach legs must be a gate highway; and
4. The intersection must be signalized and have sufficient accident data for analysis.

The intersection selection process began with establishing a preliminary list of all intersections meeting these criteria. From this preliminary list of candidate intersections, 40 were chosen for in-depth study. A detailed description of this selection process is given in the following paragraphs.

### Candidate Intersections

The preliminary list of candidate intersections was collected in two ways. The first method was to obtain a list of candidates through correspondence with signal design and operation engineers from the 12 Caltrans districts; a survey was sent to the districts to create this list. The survey responses yielded a list of approximately 80 candidate intersections. These intersections represented all Caltrans "types" of intersections: rural, suburban, and urban.

The second method of obtaining HSISI candidates involved a computer search. The computer search extracted data from the TASAS data base owned, maintained, and operated by Caltrans. The search in the statewide file was prompted with the keywords "rural," "outside city," and "signalized intersections." The computer compilation revealed 54 intersections not included in the district survey responses. Each district was contacted to determine if these additional intersections fit the criteria previously stated. Intersections that did fit these criteria were added to the preliminary list, and those that did not qualify (i.e., speed zones less than 50 mph) were discarded. From these two methods the preliminary list of 94 candidate intersections was established.

### Accident Index

The next step in the selection process was to reduce the preliminary intersection list to include 40 sites as specified in the scope of the study. Forty intersections provided a sufficient data base while still being within the resources available to the project. Since the goal of the analysis was to determine how variables affect safety at HSISIs, locations with high and low accident rates were chosen for analysis so worst- and best-case intersections would be included in the sample.

An accident index was created by taking the candidate intersection's ratio of actual accident rate to expected accident rate. The expected accident rate is a Caltrans-determined rate considering average accident rates for intersections of the same type that are classified by number of lanes, type of terrain, average highway speed, and location, that is, two-lane highway in rolling terrain with approach speeds greater than 55 mph in a rural location. The candidate intersections were listed by descending accident index values for both rural and suburban classified intersections. For the 40 chosen sites they included the 20 intersections with the highest index and 20 with the lowest index. An equally important factor in yielding statistically meaningful results was to use sites with sufficient accident histories. The time frame for the accident rates ranged from 2 to 8 years. This long time frame helped reduce the regression-to-the-mean problem. All intersections with less than 2 years of signalized accident data were discarded. There was one exception to this. The intersection at Highway 99 and Garner Lane in Butte County with 1 year of accident data was added to the 40 chosen intersections to make the total used in the study 41. Its nearness to California State University campus in Chico was valuable during preliminary development of the data base, since short trips to the site aided in development of the field data collection procedures.

### Location Classification

The last criterion to consider for intersection selection was location classification. According to the TASAS data base, there are three classifications of intersection location: rural, suburban, and urban. These classifications are based on nearby population densities and proximity to city limits. Since our preliminary list of 94 candidate intersections did not contain 41 rural classified intersections with sufficient signalized data, we had to consider other classifications. The project's goal was to analyze isolated intersections, so urban classified intersections were discarded, even though they might have met the approach speed criteria. The remaining rural and suburban classified intersections were used to devise the final list. The only changes made to this list were when field visits revealed that the intersection fell short of the necessary criteria (e.g., all approach speeds below 50 mph).

### Final List

Creating a comprehensive list of variables to describe adequately the safety features of the 40 HSISIs was an essential part of the project. An effort was made to collect all data from reliable sources. Most of the data were collected from signal design and operations offices in the 12 Caltrans districts, Caltrans headquarter offices in Sacramento, statewide county public works offices, and field visits to the chosen intersections.

The data base consisted of information describing 41 intersections across California. Each intersection had from one to three accident periods used for analysis; no accident period was longer than 6 years or shorter than 2 years. Accident periods were chosen to start and end with a change in conditions at the intersection. A unique variable was created within the data base for use in the statistical analysis: the approach accident rate. Its units are total number of accidents involving vehicles on that approach per million entering vehicles on that approach. Left-turn, rear-end, right-angle, fatal, and injury accident rates were similarly calculated on an approach basis and included in the data base. When calculating approach accident rates, accidents involving vehicles on two approaches (i.e., right-angle and left-turn accidents) will be counted twice, once each on the two concerned approaches. This method of calculation results in one intersection accident's being counted on two approaches. This occurs for left-turn and right-angle accidents only, because rear-end accidents involve vehicles on one approach only. Caution was therefore used in using approach accident rates, and comparisons between approach accident rates and intersection accident rates are not useful. One record of data represents one time period for a particular approach at an intersection. There were 6 three-leg intersections and 35 four-leg intersections. This data amounted to 271 records, each with 102 fields. Each record contained data on one approach to one HSISI over a period during which geometric, traffic control, and operating conditions remained unchanged.

Each record contained fields arranged into the following groups of data: intersection location information, intersection accident data, approach accident data, signalization data (by approach), and signing and lane marking data. All collected data were tallied onto field collection sheets and manually transferred into a dBase III Plus file.

Also defined was a high-speed approach. A variable was created in the data base that contained the values of 1 for a high-speed approach and 0 for a non-high-speed approach. An approach was considered high speed if any of the following criteria were met: (a) it had an observed mean speed of 45 mph or greater; (b) it had an observed 85th-percentile speed of 50 mph or greater; (c) it was a state highway approach with no posted speed limit; or (d) it had a posted speed limit of 50 mph or greater. Exceptions occurred when none of these criteria was met, but the site visit revealed an approach with high-speed characteristics, that is, a rural location with no traffic control within 5 mi of intersection and high-speed traffic.

Following are statistics summarizing the accident history of the 41 intersections in the data base. There were 271 approach accident periods with 1,918 total accidents. There were 19 fatal, 795 injury, 1715 multivehicle, and 203 single-vehicle accidents. There were also 282 accidents in wet conditions and 457 nighttime accidents. Types consisted of 402 right-angle, 662 rear-end, and 326 left-turn accidents. A total of 1,395 persons were injured, and 21 persons were killed.

## STATISTICAL ANALYSES

Standard statistical techniques were used and included difference in means test, difference in proportions test, analysis of variance, simple regression analysis, stepwise regression analysis, and Pearson Type III correlation analysis. For the last four analyses, SAS, which read the dBase file, was used. The data were assumed to meet the requirements for the statistical tests (i.e., normally distributed variances about the mean for regression analysis and hypothesis testing). The intended use of regression analysis was to aid in the identification of relationships between variables and accident rates. Because the use of regression was limited to linear models, better, nonlinear models could probably be developed that would describe better the relationship between some variables and approach accident rates. Since our project goal was not to predict accident rates, the regression models discussed and presented should not be used in this manner. The regression models are, however, useful in establishing that a relationship does in fact exist between an independent variable and the approach accident rate. The level of significance used throughout the statistical analysis was 5 percent.

A word of caution should be given concerning the use of comparative analysis (hypothesis testing) for data analysis. Comparative analysis will reveal statistically significant differences between groups of data. These analyses, however, do not necessarily suggest or determine the reason for these differences. Great caution must be used in assigning the cause for these differences. If care is not taken, some invalid conclusions could be drawn about the data, conclusions that are in fact statistically supported but not based on sound judgment. An example of this might be concluding that advance warning signs on approaches cause accidents. Suppose that statistical analysis revealed significantly higher accident rates on approaches with advance warning signs than approaches without advance warning signs: to assume that the signs caused the higher accident rates, when more likely the accident rates were high before the sign were installed, and the signs were ineffective, would be a poor assignment of causality even though it is statistically supported.

The first stage of the statistical analysis was to do a Pearson Type III correlation analysis between all variables in the data base. This exposed all of the significant correlations that deserved attention of further statistical analysis. The next stage of the analysis was to compare HSISIs in California results to previous research findings. This is done later in this paper. Similarities between variables in this study with previous findings validate the accuracy and completeness of the data base.

Analyzing intersections by approach may reveal relationships that may not be apparent when analyzing an intersection as an entity. For example, it is possible that an intersection with three high-speed approaches could have one approach with many accidents and two approaches with few accidents, therefore appearing to be an average intersection of that type. If higher-than-average accident rates could be determined on an approach basis, then these approach types could be identified for individual attention. The results of these statistical tests previously mentioned are discussed in the following.

### Significant Correlations

One of the first steps in the statistical analysis was to do a Pearson Type III correlation, at a 5 percent level of significance, on all data base variables versus approach accident rates. These correlations were first done on all 271 approaches and then again on the 190 high-speed approaches. These correlations were used to identify the variables for further analysis with the results contained in Tables 1 and 2 and briefly discussed later.

### Advance Warning Flashers

An advance warning flasher (AWF) in this study was considered to be a single entity consisting of an advance warning sign such as Signal Ahead in conjunction with at least one 12-in. flashing yellow beacon. An advance warning sign (AWS) is the same as an AWF but does not contain the flashing yellow beacon. Approaches in this study had the following groups represented: no advance warning; an AWF; an AWS; and both an AWF and an AWS at different locations on the approach.

- High-speed approaches with AWFs had significantly lower total, left-turn, right-angle, and rear-end approach accident rates than those without AWFs.
- High-speed approaches with AWFs had significantly lower ratios of nighttime accidents than those without AWFs,
- High-speed approaches without AWFs had no significant difference in accident rates compared with non-high-speed approaches without AWFs.
- The number of flashing lights per AWF sign, the distance from the intersection centerline to the AWF, and the location of the AWF (one side, both sides, or suspended above the roadway) did not have a significant effect on approach accident rates on high-speed approaches.
- For the location of flasher variable—on one side of the roadway only, both sides of the roadway, and suspended above the roadway—there were no significant differences between the approach accident rates among any combination.
- High-speed approaches with AWFs and AWSs did not differ significantly from high-speed approaches with AWFs, but without AWSs.

### Advance Warning Signs

The statistical analysis conducted several tests to evaluate the effectiveness of AWSs.

- Accident rates on non-high-speed approaches with AWSs were significantly higher than non-high-speed approaches without AWSs.
- Similarly, high-speed approach accident rates on approaches with AWSs are significantly higher than approach accident rates on high-speed approaches without AWSs.
- A comparison between high-speed approaches with AWFs and AWSs versus high-speed approaches with AWSs only revealed no significant difference as well.
- Approach accident rates for high-speed approaches with AWSs only were significantly higher than accident rates on high-speed approaches without either AWSs or AWFs.

Table 3 shows the mean approach accident rates, standard deviations, and sample size for each of the groups. Approaches are listed according to increasing complexity of advance warning device.

### Detector Placement

The detector setback—distance from the loop detector to the center of the intersection in feet—was one of the data base variables. Regression analysis showed that a more significant model results when considering high-speed approaches rather than all approaches. Table 2 shows the regression analysis for the model approach accident rate as a function of detector setback.

### Left-Turn Movements

Two variables in the data base pertained to left-turn movements. One was the observed number of left-turn lanes present on the approach and the other was the presence or absence of a separate left-turn phase. Of the 190 high-speed approaches, 126 approaches had left-turn lanes and left-turn phases, 40 approaches had neither lanes nor phases, and not 1 approach had a left-turn phase without a lane. Looking at high-speed approaches only, there were significant correlations between both variables and all types of approach accident rates, that is, left-turn, right-angle, rear-end, and total (approach). About 90 percent of the approaches with left-turn lanes had a separate left-turn phase also. Considering only the two left-turn independent variables, stepwise regression revealed that for high-speed approaches the presence of the left-turn phase variable was the only variable significantly correlated to approach accident rates.

A test of the difference in means between the approach accident rates for two groups was compared. Approach accident rates on high-speed approaches without separate left-turn phases were significantly higher than approaches with

**TABLE 1 Summary of Test of Difference in Means**

| Control | MAAR | SD | n | Range | Signif Diff | LoS% |
|---|---|---|---|---|---|---|
| w/o AWF | 2.54 | 2.95 | 99 | 0.1 - 12.9 | w/o > w/ | 0.02 |
| vs | | | | | | |
| w/ AWF | 1.20 | 1.15 | 91 | 0.0 - 7.28 | | |
| | MLTAR | | | | | |
| w/o AWF | 0.71 | 1.37 | 99 | 0.1 - 12.9 | w/o > w/ | 0.02 |
| vs | | | | | | |
| w/ AWF | 0.21 | 0.35 | 91 | 0.0 - 1.78 | | |
| | MRAAR | | | | | |
| w/o AWF | 1.05 | 1.73 | 99 | 0.0 - 11.1 | w/o > w/ | 0.03 |
| vs | | | | | | |
| w/ AWF | 0.41 | 0.51 | 91 | 0.0 to 2.42 | | |
| | MREAR | | | | | |
| w/o AWF | 0.37 | 0.34 | 99 | 0.0 - 1.85 | w/o > w/ | 0.44 |
| vs | | | | | | |
| w/ AWF | 0.25 | 0.26 | 91 | 0.0 - 1.17 | | |
| | MPN/D | | | | | |
| w/o AWF | 0.43 | 0.44 | 99 | 0.0 - 2.50 | w/o > w/ | 3.14 |
| vs | | | | | | |
| w/ AWF | 0.30 | 0.35 | 91 | 0.0 - 2.50 | | |
| | MAAR | | | | | |
| High-speed approaches | 2.54 | 2.95 | 99 | 0.1 - 12.9 | NO | 25.5 |
| vs | | | | | | |
| Non high-speed approaches | 2.27 | 2.46 | 81 | 0.0 - 13.3 | | |
| | MAAR | | | | | |
| w/ AWF & AWS | 1.57 | 1.17 | 14 | 0.18 - 3.3 | NO | 9.68 |
| vs | | | | | | |
| w/ AWF | 1.13 | 1.14 | 77 | 0.0 - 7.3 | | |
| | MAARNH | | | | | |
| w/ AWS | 2.64 | 2.53 | 43 | 0 - 11.6 | NO | 7.35 |
| vs | | | | | | |
| w/o AWS | 1.85 | 2.34 | 38 | 0 - 13.3 | | |
| | MAAR | | | | | |
| w/ AWS | 2.65 | 2.92 | 99 | 0.1 - 12.9 | w/o < w/ | 0.01 |
| vs | | | | | | |
| w/o AWS | 1.09 | 1.07 | 91 | 0.0 - 7.28 | | |
| w/ AWS | 2.83 | 3.10 | 85 | 0.1 - 12.9 | NO | 6.18 |
| vs | | | | | | |
| w/ AWS and w/ AWF | 1.57 | 1.17 | 14 | 0.18 - 3.34 | | |
| | MAAR | | | | | |
| w/ AWS | 2.83 | 3.10 | 85 | 0.1 - 12.9 | w/o < w/ | 0.87 |
| vs | | | | | | |
| w/o AWS & w/o AWF | 0.84 | 0.48 | 14 | 0.21 - 1.52 | | |
| | MAAR | | | | | |
| w/o Left-turn phase | 3.61 | 3.37 | 64 | 0.21 - 12.9 | w/o > w/ | 0.01 |
| vs | | | | | | |
| w/ Left-turn phase | 1.04 | 0.77 | 126 | 0.00 - 4.28 | | |
| | MREAR | | | | | |
| w/o Left-turn phase | 0.41 | 0.39 | 64 | 0.0 - 1.85 | w/o > w/ | 0.26 |
| vs | | | | | | |
| w/ Left-turn phase | 0.26 | 0.25 | 126 | 0.0 - 1.17 | | |
| | MLTAR | | | | | |
| w/o Left-turn phase | 1.08 | 1.62 | 64 | 0.0 - 7.84 | w/o > w/ | 0.01 |
| vs | | | | | | |
| w/ Left-turn phase | 0.16 | 0.23 | 126 | 0.0 - 1.11 | | |
| | MAAR | | | | | |
| Flat median | 2.28 | 2.78 | 117 | 0.00 - 12.9 | Flat > Raised | 0.05 |
| vs | | | | | | |
| Raised median | 1.30 | 1.34 | 73 | 0.11 - 7.28 | | |

Signif Diff = Statistical significance exists at 5% level
LoS% = Level of significance in percent
MAAR = Mean total approach accident rate on high-speed approaches
MLTAR = Mean approach left-turn accident rate on high-speed approaches
MRAAR = Mean approach right-angle accident rate on high-speed approaches
MREAR = Mean approach rear-end accident rate on high-speed approaches
MPN/D = Mean proportion of night to day accidents on high-speed approaches
MAARNH = Mean total approach accident rate on NON high-speed approaches
AWF = Advance warning sign accompanied by a flashing beacon
AWS = Advance warning sign

**TABLE 2 Summary of Regression Analysis**

| Indep. Variable | Depend. Variable | Parameter Estimate | t-Statistic Value | Obs | Probability F>F.05 | R2 |
|---|---|---|---|---|---|---|
| AAR | | | | 91 | 64.7 | 1.9 |
| | Type of AWF * | NS | NS | | | |
| | Location of AWF * | NS | NS | | | |
| | No. Lights per AWF | NS | NS | | | |
| | Dist to AWF | NS | NS | | | |
| AAR | | | | 190 | 0.01 | 8.4 |
| | Intercept | 3.36 | 8.69 | | | |
| | Detector SetBack | -0.004 | -4.16 | | | |
| AAR | | | | 190 | 0.01 | 26.3 |
| | Intercept | 3.61 | 200 | | | |
| | Lt-Trn Phase * | -2.57 | 67.2 | | | |
| | No. Lt-Trn Lanes | NS | | | | |
| AAR | | | | 190 | 0.17 | 6.57 |
| | Intercept | 2.46 | 117 | | | |
| | Raised Median * | -0.87 | 6.30 | | | |
| | Median Width | -0.04 | 4.91 | | | |
| AAR | | | | 271 | 0.01 | 5.67 |
| | Intercept | 2.81 | 11.5 | | | |
| | Paved Shoulder Width | -0.19 | -4.02 | | | |
| AAR | | | | 190 | 0.01 | 10.2 |
| | Intercept | 3.24 | 9.76 | | | |
| | Paved Shoulder Width | -0.27 | -4.63 | | | |
| AARAA | | | | 190 | 0.01 | 14.0 |
| | Intercept | 2.65 | 13.4 | | | |
| | No. BackPlates | -0.67 | 26.3 | | | |
| | No. Thru Faces | 0.60 | 9.4 | | | |
| | No. Mast Arm Faces | NS | | | | |
| | No. 12" Faces | NS | | | | |
| AARNHS | | | | 81 | 0.11 | 12.7 |
| | intercept | -1.62 | -1.4 | | | |
| | inter-green time | 0.85 | 3.4 | | | |
| AAR | | | | 190 | 0.15 | 5.24 |
| | intercept | 5.40 | 4.9 | | | |
| | inter-green time | -0.61 | -3.2 | | | |

AAR = Approach accident rate for high-speed approaches
AARAA = Approach accident rate for all approaches
AARNHS = Approach accident rate for non high-speed approaches
AWF = Advance warning sign accompanied by a flashing beacon
Obs = Number of observations in data set
F = F-ratio test value for model
F.05 = F-ratio at 5% level of significance
$R^2$ = Coefficient of determination of model in percent
t-Statistic = Significance test of independent variable, ≥ 1.65 to be significant
NS = Not significant for entry into model
* = Indicates a dummy variable (Present = 1, Not present = 0)

**TABLE 3 Summary of Advance Warning on HSISI Approaches**

| Type of Advance Warning on Approach | Mean Approach Accident Rate | Standard Deviation | Observations |
|---|---|---|---|
| 1. None | 0.84 | 0.48 | 14 |
| 2. AWS's | 2.83 | 3.10 | 85 |
| 3. AWF's | 1.13 | 1.14 | 77 |
| 4. Both AWS's and AWF's | 1.57 | 1.17 | 14 |

left-turn phases. The test also showed that left-turn and rear-end accident rates were also significantly higher on approaches without left-turn phases.

### Median Type and Median Width

The type of median and the width of the median in feet on an approach did not show significant correlation (level of significance less than .05) with approach accident rates for all approaches, but it did correlate significantly for high-speed approaches. A stepwise regression analysis for high-speed approaches, taking into account median type and width, revealed that the type of median explains more of the variation in approach accident rates than does the width of the median. Table 2 shows the stepwise regression parameter estimates. This is supported by Table 1, which shows the test of the difference in means between approach accident rates for two high-speed approach groups: raised medians and flat medians. The table shows that accident rates for approaches with flat medians are significantly greater than approach accident rates for approaches with raised medians.

### Paved Shoulder Width

The data base contained a variable paved shoulder width that revealed the width of the paved shoulder (in feet) measured from the right edge of the outside lane line to the edge of the paved roadway. In considering all 271 approaches, analysis of variance shows that the approach accident rate is a negative function of paved shoulder width. This shows that the wider the paved shoulder, the lower the approach accident rate. Table 2 shows the regression model results for approach accident rate as a function of paved shoulder width. The correlation of approach accident rate and paved shoulder width is greater for the 190 high-speed approaches. Table 2 shows the regression model for high-speed approaches only.

### Signal Hardware Configuration

The four variables that constitute signal hardware configuration are discussed in this section. Initial analysis showed no significant correlations among signal hardware configuration variables and the 81 non-high-speed approaches; therefore, the remaining analysis is in regard to the 190 high-speed approaches only. The four variables with their summary of statistics are (a) total number of signal faces with backplates; (b) number of through signal faces; (c) total number of mast arm–mounted signal faces; and (d) total number of signal faces with 12-in. lenses. Stepwise regression revealed that the most significant variables that affect approach accident rates on high-speed approaches are the total number of signal faces with backplates followed by the number of through faces on the approach. Neither of the remaining two variables was significant enough for inclusion in the model. Table 2 shows the final model's estimated statistical parameters. The model shows an inverse relationship between the number of signals with backplates and approach accident rates. The model also shows a direct relationship between the number of through faces on the approach and approach accident rates.

### Intergreen Time

An intergreen time variable equal to the sum of the yellow clearance interval plus the all-red time was created from the data base. A correlation analysis was run showing Pearson correlation coefficients between approach, left-turn, right-angle, and rear-end accident rates versus intergreen times. This correlation was run on three groups: all approaches, non-high-speed approaches, and high-speed approaches. Table 4 shows Pearson Type III correlation coefficients and their corresponding levels of significance. The results indicate a significant negative correlation between intergreen time and approach accident rates on high-speed approaches, a significant positive correlation between intergreen time and approach accident rates on non-high-speed approaches, and relatively little correlation between intergreen time and approach accident rates for all approaches. Two models with approach accident rate as the dependent variable and intergreen time as the independent variable were developed for high-speed and non-high-speed approaches. The results of these two models are shown in Table 2, which shows that approach accident rates are a negative function of intergreen time for high-speed approaches and are significant. It also shows that approach accident rates are a positive function of intergreen times for non-high-speed approaches and, again, are significant.

## DISCUSSION OF RESULTS AND CONCLUSIONS

The purpose of this section is to present and discuss the results of the statistical analysis, to compare the results to literature

**TABLE 4 Pearson Correlation Coefficient Values for Intergreen Time over Level of Significance**

| Inter-Green Time On: | Approach Accident Rate | Left-turn Accident Rate | Right-angle Accident Rate | Rear-end Accident Rate |
|---|---|---|---|---|
| High-speed approaches $n = 190$ | **-0.23*** 0.15% | **-0.26** 0.03% | **-0.23** 0.14% | **0.10** 15.7% |
| Non high-speed approaches $n = 81$ | **0.36** 0.11% | **0.20** 7.80% | **0.35** 0.16% | **0.26** 2.06% |
| All approaches $n = 271$ | **-0.06** 31.2% | **-0.11** 6.57% | **-0.07** 23.5% | **0.17** 0.53% |

*Pearson Correlation Coefficients in Bold Type; given in percent.

search findings, and to suggest which improvements to HSISIs might reduce approach accident rates. All of the following suggestions are based on the assumption that improvements should be considered on high-speed approaches with substantially higher than average approach accident rates over a sustained time period. Approaches with approach accident rates near or below average cannot be expected to experience accident rate reductions from suggested improvements. Approaches that benefit from improvements will most likely see a decrease in the approach accident rate over time.

AWFs seem to improve the safety of an approach to an HSISI, which is in agreement with Eck and Sabra, who found the flashing Red Signal Ahead and its variations the most effective advance warning signs (*11*). Because of these intersections' isolation, motorists tend to drive with less attention on the roadway than they would where a series of signalized intersections exists. Apparently, a flashing light draws the attention of drivers.

AWSs with no flasher have traditionally been installed on approaches to intersections with high accident rates. According to Lyles, the traditional "black cross" sign and the Vehicles Entering sign are less effective AWSs than signs accompanied by a flasher (*12*). Similarly, our analyses indicated that AWSs had little affect on approach accident rates, which may be due to the excessive use of warning signs. This excessive use may reduce a sign's impact on drivers. It was assumed that advance warning devices were installed on approaches where higher-than-average accident rates either existed or were expected and would explain the low approach accident rates associated with approaches with no advance warning.

In 1982 Barnack found that rear-end and right-angle accidents decrease as the distance from the detector to the intersection increases (*3*). Our results support those findings for approach accident rates, most significantly the approach right-angle accident rates followed by left-turn rates. The rear-end accident rates appeared to be unaffected.

An intergreen variable defined as the sum of the amber and all-red time was used to evaluate clearance intervals. A study in 1985 by Zador et al. determined that intersections with short clearance intervals are statistically correlated with higher-than-average accident rates (*9*). Similarly, our data showed that intergreen interval lengths correlated negatively with approach accident rates.

The two variables in the data base dealing with left-turn movements on an approach were the presence or absence of a left-turn phase and the presence or absence of a left-turn lane. This analysis suggested that the presence of a separate left-turn phase appeared to reduce approach accident rates at HSISIs, which is consistent with the findings of Agent (*4*). He found that left-turn phasing tends to reduce accidents between left-turning vehicles and opposing traffic. Rear-end accident reduction, directly associated with the existence of a left-turn lane, was also lower; whereas left-turn accidents, more related to the existence of a phase, were observed to be less frequent as well. David and Norman recommended that left-turn lanes should be added to an intersection only when traffic volumes warrant installation, not specifically to reduce accident rates (*5*). If a left-turn lane is added to an intersection, a separate phase for left-turns should also be added.

The type and width of a median separating directional traffic streams at an HSISI seem to influence approach accident rates. Approaches with raised medians have approach accident rates about 40 percent lower than approaches possessing medians level with the travel lanes. Wider medians generally have lower approach accident rates than narrow medians. Since widening a median may necessitate acquiring right-of-way or reducing lane and shoulder widths, installing a raised median may be a better choice for modifying medians. Installing a raised median in combination with a separate left-turn lane is attractive, because it would provide a protected left-turn pocket and would also maximize the modification effort. Installing a raised median may cause problems in colder climates where snow removal is frequent.

The paved shoulder width within sight distance of the intersection appeared to be directly related to approach accident rates. This may be an effective accident countermeasure because widening the paved shoulder may increase sight distance on an approach, and it is an easier approach modification for most rural intersections than some other improvements would be. Although not directly related to shoulder widths, providing adequate sight distance on an approach to an intersection was recommended by Hanna et al. (*7*), David and Norman (*5*), and Agent (*4*).

Signal equipment features, the number of backplates, and the number of through faces on the approach were all related to the approach accident rates at HSISIs. The addition of backplates must make signal face indications more visible to the oncoming driver, because they apparently reduced approach accident rates. Agent stated that highly visible signal heads are an important feature for a safe intersection. Approach accident rates seemed to increase as the number of through faces on the approach increases. An explanation for this may be that high numbers of through signal faces have been installed at complex intersections at which approaches tend to have higher accident rates; consequently, a positive correlation may be expected. Therefore, nothing conclusive can be said about the number of signal faces on an approach.

In conclusion, several actions should improve safety at HSISIs. The following actions should bring about reductions in approach accident rates at HSISIs identified as high accident locations. These issues should also be considered for new signal installations.

1. AWSs with flashing beacons are the most effective type of advance warning device.
2. A flashing beacon should be added to an AWS to improve its effectiveness.
3. Detectors in advance of intersections should be provided.
4. Signal heads should have backplates.
5. A separate left-turn phase should be added to an approach with an existing left-turn lane, and a left-turn lane should be installed only if justified by left-turn accidents or traffic volumes.
6. Wide, raised medians should be provided.
7. Widening the paved shoulder may be an effective improvement to use in conjunction with installation of an AWF or a left-turn lane and phase.
8. By providing channelization, left-turn phasing, and medians the roadway will take on the appearance of an urban street. Signals on urban streets are expected by motorists.
9. A demonstration project should be implemented on higher-speed approaches with higher accident rates to verify these conclusions.

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# DISCUSSION

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It is indeed a great temptation when confronted with an accident data base with more than 100 data elements to run the data through a gauntlet of statistical tests in order to ascertain some relationship between the data elements and the causes of accidents. The authors are not the first—nor, unfortunately, will they be the last—to analyze mechanically large numbers of accident data without apparent understanding of the relationships of the data elements to each other (and not just the physical data elements) and without understanding of the quality control problems of even the finest data bases. They then compound their analysis by considering only the best- and worst-case scenarios in their study design, thus introducing a considerable bias into their analysis. Some of the recommended practices in the conclusion of the study represent sound engineering practice, but others—such as detector placement and use of raised medians—are supported only by a mechanical relationship of accident data elements and do not necessarily represent sound practice.

This discussion focuses on two main areas of concern: (a) the flaws in the study design and (b) the larger issue of using accident data in safety and operations research.

## STUDY DESIGN

A common flaw in most safety analyses is that the study is focused on the "high accident locations" since those are usually identified as the problem. Much work has been done in recent years on the problem of regression-to-the-mean and its effect on accident studies. The use of longer analysis periods lends a little, perhaps, to the stability of the data, yet the number of accidents is still a statistic and that statistic is still subject to regression-to-the-mean if the highest (or lowest) accident sites are used in the analysis. The authors mention that the 41 intersections (44 percent of the candidates) used in the study represent 93 percent of the range of intersection indexes and therefore represent good coverage for the purposes of the study. Actually, this means that only 7 percent of the range of intersection indexes represents 56 percent of the candidate intersections. If the middle group had been used instead of the two extreme groups, the study intersections might have presented a more representative view of relationships between features and safety, assuming that it is possible to establish this relationship through accident data.

The accident data used in the analysis span different time periods for each approach or approach condition and cover an overall range of 11 years. A lot of things can happen over 11 years. Accident reporting climates and policies change; vehicle designs change; driver expectations and driver habits change; traffic operations and levels of congestion change—some very quickly in a state such as California. The authors took some of the physical changes into consideration by creating a new approach record when there was a change in one of the variables under study. This analysis method assumes that only those variables under study affect safety at the intersection. The other artifacts of time, however, have a much greater effect on the number of reported accidents than any of the physical elements of the intersection.

There are 271 accident approaches (records) used in the study. These contain 1,918 reported accidents, or only about 7 accidents per record. This means that there are only one or two of any particular accident type (e.g., right angle, left turn) per average approach. There is, however, no truly average intersection. The study base contains the best and the worst intersections, so there is a fairly large difference in the number of accidents per each approach record. Many of the best-case intersection approaches probably have only one or two accidents and have no left-turn or right-angle accidents, for example. These zero-case scenarios are associated with the physical conditions at the intersections and the associations are assumed to have a cause-effect relationship. In fact the relationship is due to statistical artifacts or other environmental circumstances.

The list of study intersections in Section 4.0 was quite revealing. One factor that stood out at a glance was the apparent difference in the number of years that the best- and worst-case intersections had been signalized. The 20 highest accident intersections (omitting the one that had been included for convenience) had been signalized for an average of 12 years. The lowest accident intersections had been signalized for an average of 7½ years. This implies that there may be a vast difference in the operational and environmental factors for each group, which have a far greater impact on safety than the design factors being studied.

In the discussion of results and conclusions, the authors recommend practices that they believe are supported by their study. Some of these practices are supported by other research or by sound engineering judgment. Two of the recommendations, however, are somewhat troublesome. The authors recommend the installation of a raised median when a left-turn lane is added. Many highway agencies have had bad experiences with curbs and raised medians in high-speed areas. Raised curbs violate the clear roadside concept and can cause vehicles to vault and overturn when vehicles strike them at high speeds. Perhaps the relationship between flat medians and increased accidents was due to higher accident rates in high-volume areas with narrow or painted medians. The environmental and operating conditions for those approaches with raised medians are probably much different than those with flat medians. The authors state that "caution should be used" in drawing conclusions from the analysis when the results don't make sense (e.g., increased accident rates where advanced warning signs are present as opposed to where they are not). Perhaps caution should also be used in drawing conclusions from the data even when they do not necessarily violate common sense. The authors also recommend the placement of detectors "as far from the intersection as design standards will allow" whatever that means. In general, detectors on high-speed approaches are indeed farther from the stop line than detectors on low-speed approaches. Detector placement is governed primarily by approach speed, detector design and controller settings of vehicle extension times, passage times, and intergreen times. The authors suggest that moving detectors back 100 ft can reduce approach accident rates by 10 percent. I would suggest that "caution should be exercised" in signal design before accepting this premise, which is based solely on some mechanical correlation of accident data elements.

### GENERAL USE OF ACCIDENT DATA

About the only thing one can conclude for certain when looking at a computer printout of accident data at an intersection is that some set of traffic events occurred somewhere, sometime. Hopefully, the events that are coded and recorded for the location of interest did actually occur there and not somewhere else. The problems of accident data quality are nearly universal and are too numerous to be mentioned here. In my own considerable experience in making accident studies, I found it not just useful but absolutely essential to pull hard copies of the reports from the microfilm files in order to make an intelligent assessment of the possible safety problem at a location. If the authors intend to do follow-up studies as they indicate, I would suggest that they also pull copies of the reports and reconstruct a data base that has been sanitized of as many reporting and recording errors as is possible to find. They may be astounded to find how many errors reside in accident data bases that are generally regarded as being complete and of high quality.

Some researchers have attempted to find out how many accidents go unreported. No one has come up with a definitive estimate yet, but strong evidence suggests that most accidents are never reported to anyone. This, of course, can lead to a great deal of bias in reported accidents. How well do accident data bases represent the real safety picture? What kind of meaningful conclusions can be drawn from a data base that is heavily biased even before the study begins? In fact, the more basic question to be asked is, How well does accident frequency or rate represent the relative safety of a location? Accidents at any location must be considered as very rare events when compared with the number of vehicles passing that location. Accidents are usually due to driver inattentiveness, impairment, inexperience, or plain bad judgment. The accident reports often do not reflect that, and traffic control devices and geometric design problems can become convenient methods of avoiding culpability for the accident. No one will argue that improved signal conspicuity is a bad idea, but the safety relationship between signals with 8-in. lenses and those with 12-in. lenses, or between signals with and without backplates, is very difficult to assess and probably cannot be assessed properly through accident studies.

Because the inadequacies of accident data are painfully apparent, much attention has been given to "accident surrogates," most notably, traffic conflicts. There have been two main drawbacks in the United States to the more extensive use of these surrogates. One reason is that the data must be collected, and accident data are readily available in neat computer format. The other reason that practitioners have been reluctant to use surrogates is that good strong relationships between the surrogate and the hard accident data frequently do not exist. The safety community is guilty of being locked into an accident mind-set. We are all concerned with the terrible toll of traffic accidents, and political pressure is brought to do something to reduce the number of accidents. We collect and analyze accident data to see how well our efforts are paying off. I suggest that reported accidents have only a very casual relationship to safety and that operational measures that we regard as surrogates are far better indicators of safety than accidents. In fact, I would even venture that operational measures are the true indicators of safety and that accidents are really the surrogates that are difficult to correlate to safety.

## DISCUSSION

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Laboratory and field studies serve complementary purposes and should be performed with program leadership in concert when affordable. This paper is a field study. The authors take on self-leadership by using the results from some excellent research literature. There is no reference to laboratory studies, probably because there is not much of substance or quantity to refer to; perhaps this is a problem that we have always had in highway research programs.

Some criticism of field studies appears to come from a laboratory orientation, and vice versa. Because of the very nature of field studies, variations among sites in the same category, continuous changes in site conditions over time, restrictions in the acceptability of candidate sites to any study, availability of comparable data, the accessibility of relevant and sensitive measures of performance, and the limited resources available for any comparative study, an academic laboratory approach to field studies should never be considered—and was not in this case.

The general correlational approach used is a good one, and it is apparent that the authors have a better-than-average feel for the advantages and disadvantages of this analytical approach. Multiple regression techniques and bias-checking techniques may also have been of help.

The use of accidents only to represent safety performance is unfortunate, given the many studies available that have successfully employed more active, sensitive, and controllable measures of effectiveness such as erratic maneuvers, approaching speed profiles, speed variances, and encroachments. However, credit must be given for the introduction of a new safety indicator, intuitively derived—the intersection approach accident rate. It appears that the target audience is the administration level, and that this is a usual effort to struggle to make something of accident measures.

When accidents are used as a measure, original reports should be used. Reports should be read and culled. Computerized accident information should not be used for many reasons, two of them being the annual instability of the category definitions and the high error rates inherent in using predetermined categories for ex post facto purposes. The paper does not state how it addressed this problem.

Administrators have traditionally regarded accidents as the ultimate measure of performance of safety programs and have insisted on the use of accident measures in many statistically based comparative studies in the field. However, most researchers who have used accidents in comparative studies and have studied quasiexperimental design and analysis should view accident measures as the lesser of many variables that can be used to show a relationship. Accident measures continue to be used because of the reader market for this information, not because of their inherent value. The disadvantages of using accidents in measures of performance are too numerous to discuss here. However, it should be said that a $1 million analysis in each project more often than not would overcome the more serious defects inherent in accident data collection.

Accident measures used only to describe the problem may play a practical and useful part in better understanding the general nature of conditions at intersections.

# AUTHORS' CLOSURE

Most of the excellent comments raised by the discussants were carefully considered and debated among us during the course of the project. We certainly agree that caution should be used in drawing conclusions from the data in this project and, for that matter, all such projects.

Our data base was carefully compiled, checked, and rechecked. We do not claim that it is error-free but are confident that any errors that exist are not substantial. The data were compiled from a Caltrans data base called TASAS. Accident data are coded by location by Caltrans technicians who are experienced in the process and familiar with the highway system. Location errors are minimal; probably fewer occurred than would have had an outside consultant or university employee compiled the data from original collision reports. Also, other coding errors occur—as do officer errors on the original reports.

Certainly, reporting practices and other global changes in highway safety occurred during the 11-year study period. These simply could not be controlled, and we do not claim that the statistically significant findings we make explain all the variation in accident rates. We heartily agree that one or more of the involved drivers must take most of the responsibility for most of the accidents. We did not base any of our findings on a single record of about seven accidents. Instead, we based our findings on accidents grouped for dozens of intersection approaches. The paper clearly presents sample size for each analysis. The comparisons were not necessarily made between the highest-accident-rate intersections and the lowest-accident-rate intersections. Groupings for the difference in means tests, for example, were made on the basis of an independent variable (i.e., presence of warning sign) found to be significantly correlated to accident rate in our entire data base.

We would certainly be interested in using a surrogate for accidents if one were readily available; however, we doubt that a reliable surrogate will be available anytime soon. In the meantime, we suggest that the U.S. Department of Transportation and TRB support the funding of a major comprehensive study that clearly identifies the various limitations of accident data collection and analysis methods. In addition, this study should examine the accuracy effects of these limitations and ways to minimize those effects or to modify accident data to improve accuracy. Transportation professionals would most likely appreciate such information.

We are not "guilty of being locked into an accident mind-set"; we are anxious to involve both accidents and other measures of safety in our future work. We are certainly not alone when we state that at this time we are not ready to accept outright that "operational measures are the true indicators of safety."

*The contents of this report reflect the views of the authors, who are responsible for the facts and accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the state of California or FHWA. This report does not constitute a standard, specification, or regulation.*

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*Abridgment*

# Motorist Understanding of Left-Turn Signal Indications and Auxiliary Signs

JAMES C. WILLIAMS, SIAMAK A. ARDEKANI, AND SETH ADU ASANTE

A mail survey of 6,000 drivers in Texas was conducted to assess motorists' understanding of left-turn signal indications and accompanying auxiliary signs. The principal conclusions were that (a) a green arrow should always be used for protected left turns instead of a circular green accompanied by a sign; (b) a circular red and a green arrow should not be shown simultaneously in a five-section signal head; and (c) if the red arrow is to be used in Texas, it should be accompanied by a public education program. Demographic information, including the age and years of driving, is also discussed.

Once it has been decided to provide left-turn protection at a signalized intersection, a traffic engineer must decide how to communicate this message to drivers effectively. Rules for using specific signal indications and auxiliary left-turn signs, which are intended to supplement the appropriate signal head, are provided in the *Manual on Uniform Traffic Control Devices* (MUTCD) (*1*). State manuals may provide additional guidance and options (*2*). The traffic engineer can then use a combination of signal indications and signs under the assumption that since they are already in use, the motorists must understand them.

To probe the extent of motorists' understanding of left-turn displays, a survey of specific signal indications and accompanying auxiliary signs was mailed to randomly selected Texas motorists. Following a brief overview of earlier work in this area, a description of the survey design and the analysis of responses are discussed. The results are summarized, and specific recommendations for using signal indications and auxiliary signs are made.

## LITERATURE REVIEW

Previous research in this area has focused on the development of guidelines for left-turn phasing that have typically used delay and accidents as criteria. Left-turn studies have been undertaken in Kentucky (*3*), Texas (*4,5*), Arizona (*6*), Florida (*7*), and Virginia (*8*). None of these studies addressed motorists' understanding of various signal indications and auxiliary signs.

In a recent study, more than 400 Indiana drivers were interviewed to determine their understanding of left-turn indications (*9*). The results showed that protected-only displays are better understood than permissive displays and that motorists prefer leading to lagging left turns. Two auxiliary left-turn signs were included: Left Turn Yield on Green (circular green) and Left Turn on Green or Arrow. The former sign was found to be more confusing than either the latter sign or the absence of a sign. The former sign was included in this study, but the latter sign was not because it is neither found in the MUTCD nor used in Texas.

This study investigates a much wider variety of signal indications and auxiliary signs and includes a larger sample size. In addition, two different intersection geometries are included.

## SURVEY DESIGN

When motorists are turning left at a signalized intersection, they must first decide whether the left turn is allowed during the current signal interval and, if so, whether it is protected or permitted. The questionnaire assumed that the driver was in a left-turn bay, approaching a signalized intersection. Details of the intersection geometrics and position and size (three- or five-section) of the signal heads were shown in a sketch at the top of each page of the questionnaire. There were four combinations of geometrics and signal head size and positions. Eighteen left-turn signal indications and 11 left-turn auxiliary signs were included in the survey design. A total of 40 scenarios of feasible left-turn signal–sign combinations were developed. Each questionnaire sheet included two separate signal-sign scenarios. Each driver received two questionnaire sheets, that is, four scenarios. The last portion of the survey contained several demographic questions including the respondent's gender, age, years of driving, level of education, and language spoken at home. Each scenario's geometry, signal display, and auxiliary sign are shown in Tables 1 through 4. The auxiliary sign type is shown in parentheses after the sign text; it refers to the sign illustrations in Figures 1 through 4.

## SAMPLE SIZE SELECTION

A preliminary survey was sent to 150 addresses in the Dallas–Fort Worth metropolitan area in order to fine-tune the questionnaire and to estimate the response rate. The full survey was sent to addresses in nine Texas counties (Harris, Dallas, Bexar, Tarrant, Travis, Nueces, Cameron, Lubbock, and Ector). These counties ranged in population from the highest in the state (with a population of nearly 3,000,000) to less than 100,000. Assuming a 25 percent response rate, a 95 percent confidence interval, and a tolerance of $\pm 0.025$, a sample size of 6,000 was selected (*10*). Addresses were purchased from

Civil Engineering Department, University of Texas, P.O. Box 19308, Arlington, Tex. 76019.

**TABLE 1 Description of and Responses for Type 1 Geometry**

| Signal Display | Sign | Scenario | No. of Responses | % Inconsistent | % Wrong | % Incorrect |
|---|---|---|---|---|---|---|
| Ⓖ Ⓖ Ⓖ | None | 1 | 79 | 10 | 15 | 25 |
| Ⓖ Ⓡ Ⓡ | None | 2 | 93 | 5 | 29 | 34 |
| Ⓖ Ⓡ Ⓡ | Left Turn Yield on Green ● (6) | 3 | 92 | 3 | 11 | 14 |
| Ⓖ Ⓖ Ⓖ | Left Turn Protected on Arrow Only (8) | 4 | 86 | 11 | 13 | 24 |
| ⓇG̲ Ⓡ Ⓡ | None | 11 | 79 | 2 | 21 | 23 |
| G̲ Ⓖ Ⓖ | None | 12 | 93 | 5 | 8 | 13 |
| G̲Ⓖ Ⓖ Ⓖ | None | 13 | 80 | 6 | 13 | 19 |
| G̲ Ⓡ Ⓡ | None | 14 | 80 | 6 | 14 | 20 |
| G̲Ⓖ Ⓖ Ⓖ | Protected Left on Green Arrow (2) | 15 | 91 | 2 | 6 | 8 |
| ⓇG̲ Ⓡ Ⓡ | Left Turn Signal (7) | 16 | 93 | 1 | 33 | 34 |
| G̲ Ⓖ Ⓖ | Protected Left Turn on Arrow Only (9) | 17 | 93 | 1 | 17 | 18 |
| G̲ Ⓡ Ⓡ | Protected Left Turn on Green Arrow Only (11) | 18 | 86 | 1 | 4 | 5 |

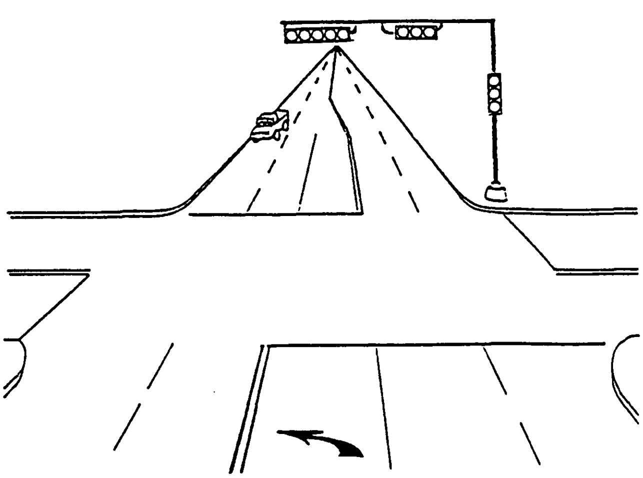

**FIGURE 1 Type 1 geometry.**

a local firm that provides mailing lists for marketing and advertising purposes.

A total of 894 surveys were returned, making for a response rate of 14.9 percent. The lower-than-expected response rate increased the design tolerance level to $\pm 0.033$ at a 95 percent confidence interval.

## RESULTS

The percentage of incorrect responses for each scenario, broken down into inconsistent or wrong responses, is shown in the tables. Typical inconsistent responses included cases in which the respondent indicated that a left-turn was prohibited but proceeded to indicate that the turn could be made on a protected or permissive basis. Another inconsistency was when the driver indicated that a left-turn could be made without specifying whether it was protected or permissive. A response was considered wrong only if it was incorrect but consistent. A discussion of indications and signs that were particularly troublesome for many drivers and a breakdown of the responses in accordance with the demographic data follow.

## DEMOGRAPHIC EFFECTS

The demographic factors that were thought most likely to affect the fraction of incorrect responses were years of driving and age. Those who had been driving for 11 to 20 years had the lowest percentage of incorrect responses (22 percent). Correspondingly, respondents 26 to 35 years old also had the lowest percentage of incorrect responses (20 percent). Most drivers in the United States start to drive in their middle to

**TABLE 2 Description of and Responses for Type 2 Geometry**

| Signal Display | Sign | Scenario | No. of Responses | % Inconsistent | % Wrong | % Incorrect |
|---|---|---|---|---|---|---|
| Ⓖ Ⓖ Ⓖ | None | 5 | 102 | 3 | 29 | 32 |
| Ⓖ Ⓡ Ⓡ | None | 6 | 83 | 4 | 27 | 31 |
| G̲ Ⓡ Ⓡ | None | 19 | 83 | 4 | 13 | 17 |
| G̲ Ⓖ Ⓖ | None | 20 | 109 | 5 | 11 | 16 |
| G̲ Ⓡ Ⓡ | Left Turn Signal (7) | 21 | 81 | 6 | 11 | 17 |
| G̲ Ⓖ Ⓖ | Left Turn on Arrow Only (1) | 22 | 82 | 6 | 14 | 20 |
| Ⓖ Ⓡ Ⓡ | Left Turn Signal (4) | 23 | 73 | 3 | 39 | 42 |
| Ⓖ Ⓖ Ⓖ | Left Turn Signal (4) | 24 | 96 | 6 | 59 | 65 |
| Ⓡ Ⓖ Ⓖ | None | 33 | 103 | 16 | 8 | 24 |
| R̲ Ⓖ Ⓖ | None | 34 | 110 | 17 | 16 | 33 |
| R̲ Ⓖ Ⓖ | Left Turn Signal (7) | 35 | 75 | 20 | 32 | 52 |
| Ⓡ Ⓖ Ⓖ | Left Turn on Arrow Only (1) | 36 | 95 | 19 | 2 | 21 |

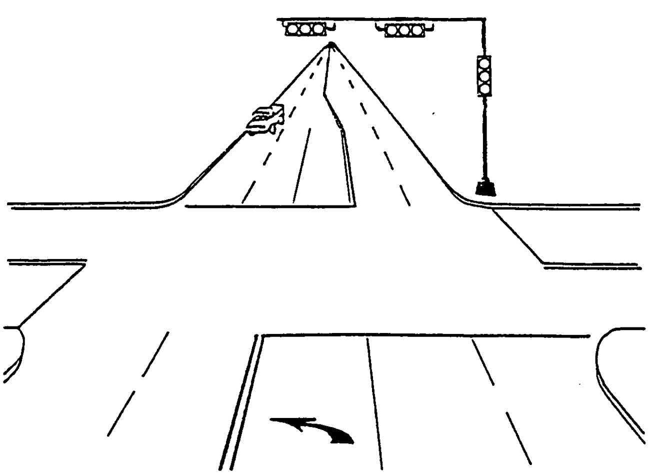

**FIGURE 2 Type 2 geometry.**

late teens, so this correspondence is not surprising. Higher percentages of incorrect responses were found for less (29 to 31 percent) and more (24 to 31 percent) experienced drivers and for younger (25 percent) and older (23 to 35 percent) drivers. Perhaps the longer people drive, the more they are exposed to and understand differing left-turn treatments. This would be true only to a point, however: drivers over 65 had the highest percentage of incorrect responses (35 percent). Often these drivers avoid congested traffic areas, because they generally do not work, and they may not be as familiar with some indications that have been in use for only a relatively small fraction of their driving life.

A statistical comparison was conducted between various age and driving experience categories by using the Waller grouping technique. Waller's test groups categories whose means are not significantly different, thus identifying those that are (*11*).

Here, the value tested for each category was the percentage of incorrect responses. All tests were done at $\alpha = 0.05$. In driving experience, if the groups with higher percentages of incorrect responses are put together, only drivers with 11 to 20 years of driving experience have a significantly lower percentage of incorrect responses. Similarly, if the categories with lower fractions of incorrect responses are grouped together, only drivers with 41 to 50 years of driving experience have a significantly higher percentage of incorrect responses.

A similar line of reasoning for the results of Waller's test on the driver age categories (again at $\alpha = 0.05$) showed that the spread in the percentage of incorrect responses between categories was large enough to allow three separate groupings, one of lower percentage of incorrect responses (ages 26 to 35 and 36 to 45), one of higher percentage of incorrect responses (25 and younger and 66 and older), and one in the middle (46 to 55 and 56 to 65).

**TABLE 3 Description of and Responses for Type 3 Geometry**

| Signal Display | Sign | Scenario | No. of Responses | % Inconsistent | % Wrong | % Incorrect |
|---|---|---|---|---|---|---|
| Ⓖ Ⓖ | None | 7 | 88 | 9 | 13 | 22 |
| Ⓖ Ⓖ | Left Turn Yield on Green ● (6) | 8 | 80 | 4 | 9 | 13 |
| Ⓖ Ⓖ | Left Turn on Green After Yield (10) | 9 | 80 | 4 | 14 | 18 |
| Ⓖ Ⓖ | Protected Left on Green (3) | 25 | 89 | 7 | 36 | 43 |

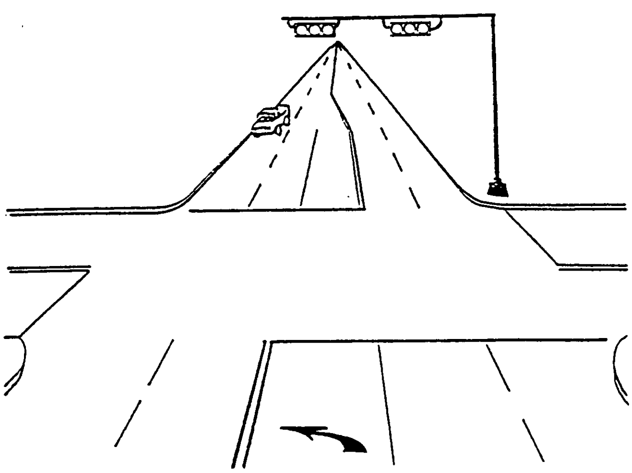

FIGURE 3 Type 3 geometry.

## DISCUSSION OF SPECIFIC SCENARIOS

The survey results concerning four separate left-turn indication issues are discussed in this section.

### Use of Circular Green for Protected-Only Left Turns

The circular green indication was used for protected-only left in Scenarios 10 and 23 through 26, and a green arrow was used for protected-only left in Scenarios 19 through 22 and 27 through 32. The percentage of incorrect responses for each scenario is shown in Tables 1 through 4. Many drivers appear to believe that the circular green indicates a permissive turn, even in the presence of an auxiliary sign. The respondents appeared to understand more consistently the use of the green arrow for protected-only left turns.

### Use of Circular Red and Green Arrow Simultaneously in a Five-Section Head

This indication is used during the dual-left portion of the cycle (when opposing lefts are turning simultaneously), and left turns are protected while throughs face red. Scenarios 11 and 16 show the circular red and the green arrow simultaneously in the five-section head, and the circular red is shown on the other mast arm–mounted signal head. Scenarios 14 and 18 show the same condition, omitting the circular red in the five-section head. The percentage of incorrect responses for these scenarios is shown in Table 1. When the circular red was not displayed, driver understanding increased, perhaps indicating that displaying both red and green in the same head confuses many drivers.

### Use of Auxiliary Left-Turn Signs

This information is somewhat more difficult to interpret. Responses were examined for scenarios with the same signal displays and geometric characteristics but different auxiliary signs. Two questions were examined: Which signs have the lowest overall level of misunderstanding? and Which signs show the greatest improvement over the no-sign case?

Figure 5 shows the various auxiliary signs examined. Sign Types 2, 6, and 11 showed the smallest percentage of incorrect responses, and all three types show improvement when the sign is added to a particular scenario. Signs 3, 4, and 7 showed the lowest levels of understanding, and, in each case, driver understanding was either the same or better if the sign was not present. Type 7 is a special case, observed only in the city of Austin, and therefore was probably unfamiliar to the great majority of the respondents.

### Use of Red Arrows

The use of red arrows was tested in eight scenarios, four using a red arrow and four using a circular red to prohibit left turns. By and large, the percentage of incorrect responses was smaller (13 to 28 percent) when a circular red was used to prohibit left turns than when using a red arrow (29 to 52 percent). This was found to be true in every category when similar mounting and sign conditions were paired. A possible explanation is that, with red arrows, drivers may be confusing the red indication (meaning prohibition) with the arrow indication (meaning movement in that direction). Thus, they may be interpreting the red arrow indication to mean that they may proceed with caution to make a permissive turn. It should, however, be noted that, since the red arrow is used in only one city in Texas (Odessa), many of the survey recipients

TABLE 4 Description of and Responses for Type 4 Geometry

| Signal Display | Sign | Scenario | No. of Responses | % Inconsistent | % Wrong | % Incorrect |
|---|---|---|---|---|---|---|
| Ⓖ Ⓖ Ⓖ | None | 10 | 84 | 2 | 48 | 50 |
| Ⓖ Ⓡ Ⓡ | None | 26 | 107 | 2 | 45 | 47 |
| G̲ Ⓡ Ⓡ | None | 27 | 96 | 5 | 4 | 9 |
| G̲ Ⓖ Ⓖ | None | 28 | 95 | 5 | 22 | 27 |
| G̲ Ⓡ Ⓡ | Left Turn Signal (4) | 29 | 103 | 10 | 7 | 17 |
| G̲ Ⓖ Ⓖ | No Turn on Red (5) | 30 | 103 | 6 | 25 | 31 |
| G̲ Ⓡ Ⓡ | Left Turn Signal (7) | 31 | 92 | 4 | 10 | 14 |
| G̲ Ⓖ Ⓖ | Left Turn Signal (7) | 32 | 69 | 9 | 14 | 23 |
| R̲ Ⓖ Ⓖ | None | 37 | 107 | 15 | 14 | 29 |
| Ⓡ Ⓖ Ⓖ | None | 38 | 87 | 7 | 6 | 13 |
| R̲ Ⓖ Ⓖ | Left Turn Signal (4) | 39 | 91 | 19 | 12 | 31 |
| Ⓡ Ⓖ Ⓖ | No Turn on Red (5) | 40 | 68 | 15 | 13 | 28 |

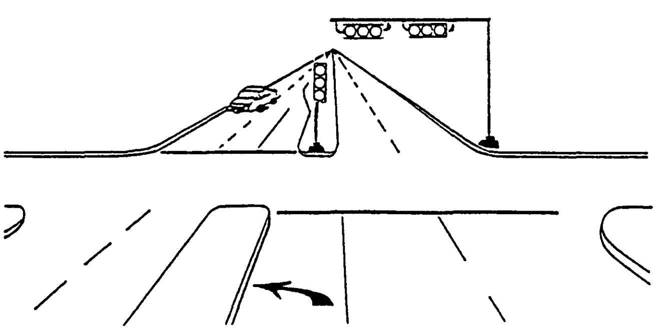

FIGURE 4 Type 4 geometry.

were likely to be unfamiliar with it, which could result in a higher fraction of incorrect responses.

## CONCLUSIONS AND RECOMMENDATIONS

It may appear alarming that such a large fraction of drivers misunderstand some of the more commonly used left-turn treatments. However, it must be kept in mind that only a single signal interval is shown in the questionnaire and that the respondent is deprived of many audio and visual clues available in the field.

On the other hand, sound engineering practice dictates that each signal indication should be self-sufficient; that is, it should convey a complete message by itself. This survey provides a good measure of this feature, and guidelines concerning the use of specific indications can be drawn from it. Ideally, motorists should understand all signal indications, under all conditions, regardless of the absence or presence of clues.

From the observations in the previous section, the following recommendations are made:

1. If red arrows are used, their use should be accompanied by an educational program. They were not as well understood as a circular red to prohibit left turns during a particular interval, but red arrows are seldom used in Texas. One advantage of the use of red arrows is that auxiliary signs are not necessary on the left-turn signal head.
2. A green arrow should always be used for protected left turns. Even when an auxiliary sign was used with a circular green intended for left turns, the fraction of respondents answering incorrectly was higher than for equivalent cases with green arrows.
3. A circular red and a green arrow should not be shown simultaneously on a five-section head. This indication is used

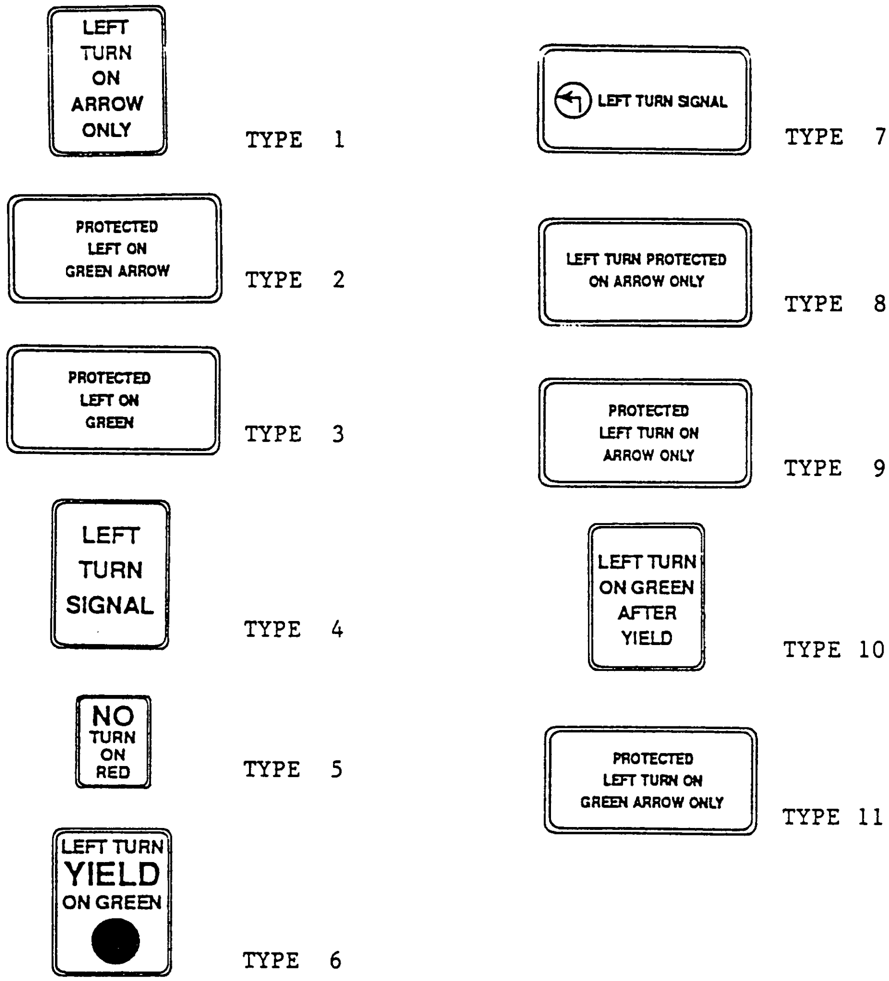


**FIGURE 5 Left-turn auxiliary signs.**

to indicate a protected left while the through traffic is allowed to go. When the circular red was removed, the fraction of respondents answering incorrectly dropped.

A recommendation for the auxiliary sign is somewhat more difficult to make. A primary disadvantage of any auxiliary sign is that it is difficult to read at night unless it is directly illuminated. Those that state that lefts were protected on the green arrow (Types 1, 2, 8, 9, and 11 in Figure 5) are superfluous, because drivers appeared to have a good understanding of the meaning of the green arrow. The indication that causes most of the confusion in this regard is the circular green when applied to left turns: does it provide for protected or permissive operation? Therefore, if a sign is necessary, one that indicates that left-turning traffic must yield on the circular green is preferred.

4. Sign Type 6, Left Turn Yield on Green (circular green), should be used, if necessary, when permissive turning is allowed. Type 10 has a similar message, Left Turn on Green After Yield, but is not as clear because neither circular green nor green arrow is specified.

## ACKNOWLEDGMENTS

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# Incident Management Using Total Stations

LESLIE N. JACOBSON, BILL LEGG, AND AMY J. O'BRIEN

Many strategies have been used to improve incident management. A novel approach to improving incident clearance when accidents require detailed investigation is described. This approach involves the use of computerized surveying equipment, called electronic total stations, for accident investigation. The use, advantages, and disadvantages of electronic total station survey equipment for expediting the investigation at serious traffic accidents are discussed. A comparison of three accident scenes where the coordinate method of accident investigation and total stations were used to measure the same incident showed that investigators can take over 70 percent more measurements per hour by using total stations. Furthermore, investigation with the total stations required only 46 percent of the time required with the coordinate method. A comparison of urban freeway accidents covering 1 year before total stations were used to 1 year during total station use showed an average time savings of slightly more than 51 min in the time to clear the scene of an incident. Accident drawings were also more accurate and could be prepared in less time.

A 1989 study of the nation's 37 largest urban areas by FHWA estimated that in 1987 more than 60 percent of all freeway congestion in urban areas was caused by incidents. Incident-related delay cost freeway users an estimated $10 billion in excess fuel consumption and motorist delay. By 2005, the estimates indicate, more than 70 percent of all urban freeway congestion will be caused by incidents, costing users nearly $64 billion (*1*).

Because of the tremendous effect that incidents have on our transportation system and the economic health of our nation, transportation professionals have focused more and more on incident management as an extremely effective tool to reduce congestion and increase safety on urban freeways across the country. Incident management is the "coordinated, preplanned use of human and mechanical resources to restore full capacity after an incident occurs, and to provide motorists information and direction until the incident is cleared" (*2*). Incident management can be divided into five basic tasks (*3*):

- Incident detection and verification,
- Incident response,
- Incident site management,
- Incident clearance, and
- Motorist information.

Many strategies have been used to improve incident management in each of these basic areas. However, few techniques have been devised to speed incident management when formal law enforcement investigation is required. This paper describes a novel approach to improving incident clearance when accidents require detailed investigation. This approach involves the use of computerized surveying equipment—electronic total stations—for accident investigation.

In a recent study of incidents in the Seattle area, researchers found that almost 40 percent of the accidents in the most heavily traveled freeway sections involved injuries (*4*). Furthermore, as the number of injuries increased, so did the length of time required to clear the accident. In general, as accident severity increases, so does clearance time. One reason for this increase is the investigation time required for certain accidents. Measurements must be taken of the accident site, and detailed information on vehicle location must be collected. By reducing the amount of investigation time, the overall clearance time for these accidents can be reduced and with it, the subsequent congestion, excess fuel consumption, and pollution emissions.

In a recent demonstration project, electronic surveying systems were used by Washington State Patrol personnel; these systems are called infrared total stations. Several manufacturers offer this equipment. Total stations allow automatic distance measuring, and the measures are fed directly into a computer for analysis. Total stations were designed for general surveying work, and surveyors have used them for many years. This paper describes the use, advantages, and disadvantages of electronic total station survey equipment for expediting required investigation at scenes of serious incidents. Even though police or state patrol are most likely to use this technology, other incident management agencies—notably, state highway agencies—may be most interested in its use because it reduces the time to clear an incident, reducing delays to motorists and minimizing the time that incident respondents are exposed to danger during an incident.

## TRADITIONAL ACCIDENT INVESTIGATION

Accident investigation is a necessary part of incident response and management. On Washington State's highways and freeways this duty falls on the Washington State Patrol. They are required to investigate accidents involving fatalities, alcohol or other drugs, and suspected felonies. They also investigate any accident that they believe will be the subject of court action. The information is used to prosecute violators; produce statistics that influence safe vehicle, pavement, and highway design; and provide evidence during litigation.

A detailed investigation requires a large number of data. Troopers determine the cause of the accident by establishing the direction of travel, speed, and any unusual movements of all the vehicles involved. They must measure the locations at which skid marks start, end, or change direction; the locations at which curves begin or end; debris patterns; gouges and

Washington State Transportation Center, Washington State Department of Transportation, 4507 University Way, N.E., Room 204, Seattle, Wash. 98105.

scratches; intersections; warning signs; and any other potentially relevant details.

There are several steps to accident investigation. After the troopers have been called to the scene, they find the important objects to be measured. Next, they measure all the physical evidence at the scene. Then they remove their equipment from the site. Finally, back at the office, they file a report and recreate the scene in a scale drawing.

Although these steps have remained fairly constant over the years, accident investigation and reporting methods have evolved. Most often today, investigators use the coordinate method. They lay down a base tape straight through the accident scene. The locations of all other objects and sites are measured as a distance perpendicular to the base tape. This method takes a minimum of two people, and usually three: one to hold the tape, one to read the measurements, and one to write the numbers and descriptions of the measured points in a field book. Traffic control people are also required when measurements are made across traffic lanes.

Back at the office, staff painstakingly recreate the scene by hand. They start by drawing the baseline created in the field. They draw all the measured points to scale in relation to that line and rely on the field book notes to tell them what the points represent. They complete the drawing with hand-drawn vehicles, trees, signs, and any other objects measured.

This method of accident investigation is accurate, but it can cause problems for the troopers and the motoring public.

### Disadvantages to Motorists

The greatest negative effects of long clearance times associated with accidents that require formal investigation are probably borne by the traveling public. Depending on the size of the incident, current methods of accident investigation normally require that lanes, or even the entire roadway, be closed so that investigators can measure the site. Depending on the size of the accident, the road may be closed anywhere from 45 min to several hours.

Many studies have shown the serious effects of freeway incidents on congestion.

- A study in California showed that for each minute of blockage during off-peak periods on freeways, 5 min of congestion can be expected. During peak periods, the effects on congestion are much greater (*5*).
- A study in Houston showed that 80 percent of all incidents reduced capacity by at least one-third, regardless of whether a lane was blocked. On a three-lane freeway, the capacity was reduced by half when one lane was blocked (*6*).
- In Seattle motorists experienced an estimated 18.4 million hr of delay in 1984, 58 percent of which was the result of freeway incidents (*7*).

This congestion has important consequences to the public. For example, congestion causes safety problems. The longer the road is closed, the greater is the probability that secondary accidents will occur in the queue.

Also important, although difficult to quantify, are the costs of driver frustration. This frustration can, in turn, compromise safety, as delayed, angry drivers may take unreasonable risks.

### Disadvantages for Responding Personnel

The traditional method of accident investigation also has disadvantages for the personnel responding to the incident. A major consideration is safety. The coordinate method requires investigators to be in the traveled roadway to take most of their measurements. Even though lanes may be closed, these measurements are often taken next to open lanes of traffic, increasing the investigators' exposure to danger.

Another major disadvantage for those responding to incidents is time. Enforcement personnel report that they are able to take about 30 to 45 measurements an hour. Depending on the severity and size of the accident, investigation may take anywhere from 45 min to 6 hr or more. The extensive time required for full investigation not only affects the traveling public and increases all incident respondents' exposure to traffic dangers, but affects the efficiency of the organizations to which the respondents belong. The longer that respondents are at an incident scene, the longer they are unavailable for other duties and the more expensive the incident becomes. In addition, an investigator may need up to 8 hr to complete a scale drawing.

Data accuracy and completeness are also issues. Investigators must take measurements at a 90-degree angle from the baseline tape. If they measure objects 30 ft or more away from the baseline, they risk unintentionally moving the tape and creating errors. Furthermore, their longest tapes are fiberglass, which is temperature-sensitive and can stretch 2 to 3 in. over 300 ft.

A final factor is convenience. For example, the tape is lightweight and may blow in the wind. The investigators may also have trouble clearing a path for the baseline, which must be flat on the ground and run through the accident site. In addition, the longest tape is 300 ft. If the site is larger than that, the troopers must set out a second baseline.

## TOTAL STATION TECHNOLOGY

The electronic total station is a combination of an electronic distance meter, which uses an infrared light to measure distance, and a theodolite, or electronic transit. Several manufacturers offer this equipment. The cost for the field equipment is in the range of $15,000, bought in any quantity (the list price is somewhat higher). Surveyors have used this equipment for about 15 years, but it has only recently been adopted for measuring accident scenes.

The electronic distance meter calculates distance by sending an infrared light to a prism on a rod, which is held on the object to be measured, and averaging the time the light takes to move to the prism and return. The infrared light replaces the measuring tape used in the coordinate method. The theodolite measures the horizontal angle from the 0 point at the baseline. Because the station has an internal level, it can also measure vertical angles.

To take measurements, an investigator places the total station at a site from which he or she can view all the objects to be measured. Because the prism is on a tall rod, the total station can measure over the tops of objects, including moving traffic. Most of the time one placement is all that is necessary. One person holds the rod with the prism on a point or object to be measured. Another person sites the total station on the prism. This simple procedure allows the station to measure distance, horizontal angle, and vertical angle simultaneously. A small window shows the measurement information while it is being collected and calculated.

The total station also includes a computer that stores the data as they are electronically collected, replacing the traditional field book. The investigator keys in a code or label for each measured point. The code is a drafting or plotting command for use with an office plotter or computer system.

In the office, the total station connects directly to a plotter to download the information for a quick, crude plot. It can also be connected to a microcomputer so that the investigator can manipulate the data within a data base or drafting program. The drafting program interprets the assigned codes and plots a diagram. It can add details of cars, trees, and other objects that have been predrawn and stored in the computer. The investigator can even produce an animated recreation of the incident.

## EVALUATION

The research team evaluated the use of total stations for accident investigation as an incident management strategy. The purpose of the evaluation was to (a) determine the effectiveness of using total stations for accident investigation, (b) determine the value of using total stations for incident management, and (c) produce information that would encourage the use of total stations as an incident management strategy.

The research approach was broken into three parts.

1. Investigation times were compared on three accident scenes at which both the coordinate method and the total station method were used.
2. Incident clearance times were compared on urban freeway accidents in the Seattle area that required formal investigation. Accidents from 1989, before total stations were used, were compared with 1991, when total stations were used exclusively.
3. Researchers estimated the benefit-cost relationship of using total stations as incident management tools.

### Investigation Time

The first evaluation method involved the investigation itself. Three accident sites were measured with the traditional coordinate method and with electronic total stations. The first accident (Site A) was a one-car fatality on a two-lane rural highway. The accident occurred at night. The driver ran off the road on a curve near an intersection and struck a roadside telephone pole. The driver died on the scene.

The second (Site B) was a two-car collision involving a police car. The accident occurred at night in a small town on a two-lane state highway. One car pulled out of a side street to turn left onto State Route 2. The driver apparently did not look to his left and was struck broadside by a police car traveling on the state highway. The accident resulted in serious injuries.

The final accident (Site C) also occurred on a two-lane rural highway at night. A county deputy was involved in the collision, and a very serious injury resulted. The driver of the civilian car crossed the centerline on a curve and struck the deputy's car head-on.

According to the Washington State Patrol detective that measured the scenes with the total station equipment, all three accidents were typical of accident scenes investigated. None of the geometrics was particularly complex. The investigation time evaluation compared two measures of effectiveness: measurements taken per hour and time to measure a scene fully. For each accident, Table 1 shows the number of measurements taken and the time required to investigate the scene fully with both the coordinate and total station methods.

From this information, the advantage of total station accident investigation is clear. A calculation based on the average of the three accidents reveals that electronic total stations require only about 46 percent of the time to investigate an accident scene that the coordinate method requires. In addition, investigators can take over 70 percent more measurements per hour with total stations than with the coordinate method.

One advantage of total stations that is not evident from these measures is the ability to provide more accurate and detailed collision and scene diagrams. In each of the three accidents listed, only the bare essentials of the accident scene were measured with the coordinate method, whereas the entire scene was measured and subsequently plotted with the total stations.

At Site A, investigators measured only part of the roadway curve, one edge stripe, four tire marks, and the final point of rest using the coordinate method. No intersection information was collected. With the total station method, they measured and plotted the entire intersection and roadway.

At Site B, similar results were obtained. Using the coordinate method, the investigators measured the site of impact, the tire marks, and the final point of rest. With the total station method, they were able to measure the entire intersection.

**TABLE 1 Comparison of Investigation Techniques**

| Accident Site | Measurements per Hour | | Time to Investigate | |
|---|---|---|---|---|
| | Coordinate Method | Total Station | Coordinate Method | Total Station |
| Site A | 32.4 | 47.0 | 150 min | 60 min |
| Site B | 30.0 | 50.4 | 90 min | 75 min |
| Site C | 24.0 | 52.0 | 150 min | 45 min |
| Average | 28.8 | 49.8 | 130 min | 60 min |

At Site C, investigators used the coordinate method to measure the entire roadway over a 300-ft section of the curve where the accident took place. Using the total station method, the investigator measured the entire curve and tangent sections on either end of the curve. A ¼-mi section of roadway was fully detailed.

To give a better understanding of the improved quality of accident diagrams produced from total station data, typical diagrams produced from both types of investigation are presented here. Figure 1 is a reproduction of an accident scene investigated with the coordinate method. Figure 2 is a reproduction of an accident scene investigated with total stations. (Figure 2 is a black-and-white reproduction of color graphics, whereas the diagrams produced from the coordinate method were drawn in black and white.) Additionally, many more types of diagram can be produced quickly and efficiently. For accidents investigated with the coordinate method, only scene diagrams were produced. By using total stations for field investigation and an office computer, investigators can easily prepare momentum diagrams, time and distance diagrams, and vehicle damage profiles, as well as scene diagrams. The accident can even be presented with animation on a computer screen.

The Washington State Patrol estimates that its investigators need about 2 hr to complete a drawing that used to take 8 hr by hand. The system creates a computerized data base of site drawings that are reusable when an accident occurs at a previously measured site. Washington State Patrol has found this feature especially useful for complex intersections.

## Incident Clearance Times

The evaluation of incident clearance times covered accidents on freeways within the Seattle urban area. These incidents have a great effect on motorist delay and overall freeway

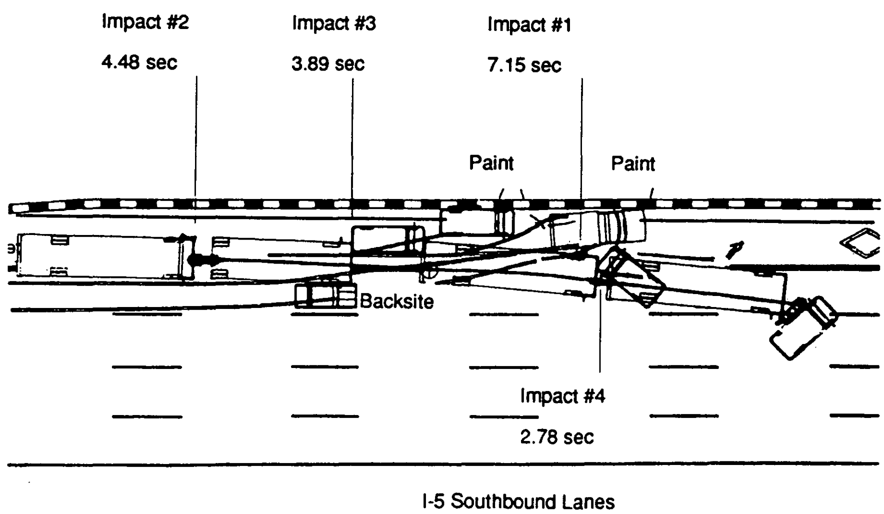


**FIGURE 1 Typical scene diagram, total station method.**

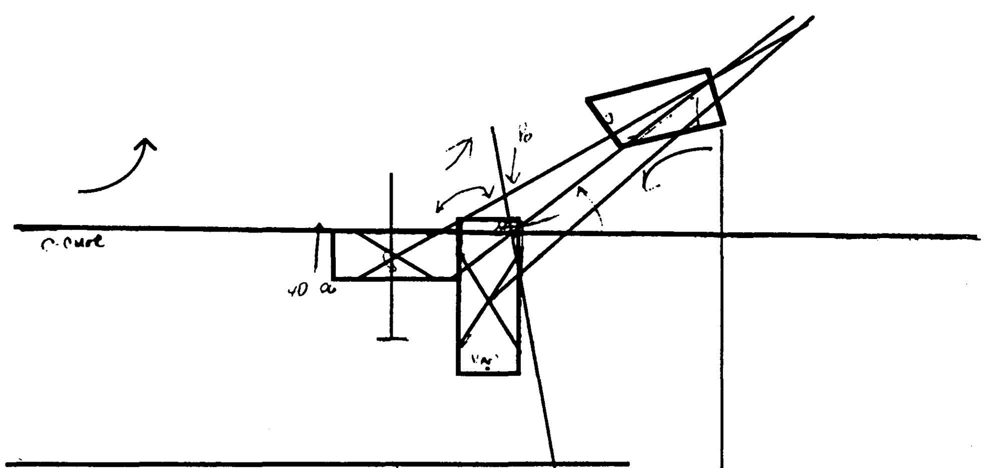

**FIGURE 2 Typical scene diagram, coordinate method.**

operations. Incident management efforts are focused on incidents on these sections of roadway.

Urban freeway accidents that were investigated in 1989, before total stations were in use, were compared with urban freeway accidents that were investigated in the first 7 months of 1991, after total stations were in use on all investigations. The difference between the time at which the Washington State Patrol was notified of the accident to the time at which the incident was cleared was compared for the two groups of accidents. Researchers had to match two separate data sources to extract the needed information. One data source was the log of all traffic investigations. This data source did not include any indication of time of notification or time of clearance, but it did provide a list of all accidents that were investigated using the coordinate method in 1989 and total stations in 1991. The Washington State Patrol's computer-aided dispatch files were used to extract the time of notification and time of clearance. The only common fields in the two data sources were date, type of accident, and location. The two data sources did not use exactly the same descriptions for location and type of accident. Therefore, researchers had to be very conservative in determining matches between the two data sources. Although about 40 urban freeway accidents are investigated each year in the Seattle area, only 20 matches were found in the 1989 data and 16 in the 1991 data.

The next step was for the researchers to look at accident type to determine whether roughly similar types of accidents were investigated in the 2 years. In this process, the researchers discovered that a relatively rare type of accident was investigated in 1991 (a combination of automobile theft and vehicular assault) that did not appear in 1989. This was the only instance of this combination in any of the data searched. All other types were found on several occasions in both years. The automobile theft and vehicular assault incident was discarded in all further evaluation.

For all remaining accidents, 20 for 1989 and 15 for 1991, clearance times were determined by subtracting the time of clearance (the time that the roadway was cleared of the accident) from the time of notification (the time the state patrol first received information on the accident). The average and standard deviation were calculated for each set of clearance times. Accidents that were investigated with total stations were cleared an average of 51 min sooner than accidents investigated with the coordinate method (131 min for total stations and 182 min for the coordinate method). The standard deviations of clearance times for the two types of investigation were equivalent (68.5 min for total stations and 69.9 min for the coordinate method). The statistical significance of the difference in the means was tested at the 95 percent confidence level. The difference was statistically significant at that confidence level.

Clearance time alone does not fully describe the benefits of the system. Total stations can be set off of the roadway, and measurements can be taken across open lanes of traffic. Lanes can be opened more quickly, reducing motorist delay. About the same time that the state patrol began using total stations, the Washington State Department of Transportation (WSDOT) began keeping track of lane closure time for every incident to which it responded. These data were not available for the years in which the state patrol used the coordinate method, so a direct comparison was impossible. However, this advantage of total stations should not be overlooked.

### Benefit-Cost Relationship

The research team conducted a benefit-cost analysis of the use of total station equipment for accident investigation. The cost of the equipment was compared with the value to the traveling public of reduced fuel consumption and reduced delay. A simple computer model of freeway queueing analysis (FREWAY) was used to determine differences in delay for accidents investigated with total stations and with the coordinate method. Traffic volumes are the primary data needed for this model.

Because of the difficulty in finding hourly traffic volumes for each accident investigated, one peak-period accident that occurred in 1991 was selected as the basis for this evaluation. The accident occurred in the peak direction southbound on Interstate 5 (toward downtown Seattle) shortly after 7:00 a.m., the height of the morning peak period, on Wednesday, May 22, 1991. Because of a hardware problem in the computer that would have accumulated traffic information for the Seattle area freeway management system, exact data for that day were unavailable. The research team used the published 1990 peak-hour volume for the accident location (*8*). To use the results directly from the evaluation of clearance time, the research team assumed the average clearance times for the total station (131 min) and coordinate method (182 min) cases. A 4-hr simulation time was chosen.

No volume data were available other than for the peak hour; however, the model required volume information throughout the incident and through the time that congestion ceased. The researchers analyzed hourly data from the same section of freeway at other times of the year and found that about 87 percent of the peak-hour flow occurred between 8:00 and 9:00 a.m., about 67 percent between 9:00 and 10:00 a.m., and 58 percent between 10:00 and 11:00 a.m. Because the researchers believed that volumes in the mid-morning were more variable than during the peak period, a conservative estimate of 50 percent of the peak flows was used for the hours 9:00 to 10:00 and 10:00 to 11:00. Table 2 shows the volumes used for the simulation.

Because no information on lane blockages was available for this incident, the research team assumed that two lanes out of five were blocked for the duration of the incident. The model then calculated the unrestricted and incident capacities of the freeway segment. Table 3 presents the results of the simulation. As can be seen in the table, the use of total stations saved nearly 7,000 vehicle-hr of delay for this accident.

There are several weaknesses in this evaluation. The actual volumes were not used. The average clearance times, rather than the specific clearance time for the given accident, were used to normalize the accident. The exact number of lanes closed during the accident was not available. Finally, only

**TABLE 2 Hourly Volumes Used for FREWAY Simulation**

| Time of Day | Traffic Volume |
|---|---|
| 7:00 - 8:00 am | 9090 |
| 8:00 - 9:00 am | 7900 |
| 9:00 - 10:00 am | 4500 |
| 10:00 - 11:00 am | 4500 |

**TABLE 3 Comparison of Simulation Results**

| Measure | Coordinate Method | Total Station |
|---|---|---|
| Total Delay | 22,211 veh-hr | 15,365 veh-hr |
| Average Delay | 51 min/delayed vehicle | 37 min/delayed vehicle |
| Delay Savings | – | 6,846 veh-hr |

local impacts were taken into consideration, even though an accident of this severity during the morning peak period would certainly affect conditions on many other links in the system. However, the research team took a conservative approach on each of these instances, so the benefits of using the total station equipment were underestimated. The simulation results should be viewed as typical savings for a representative peak-period accident.

The researchers checked the validity of these results with earlier incident research conducted by Mannering et al. at the University of Washington (*9*). The Mannering study analyzed the systemwide impacts of peak-period incidents at specific locations on the network. One such location was on Interstate 5 approximately 4 mi south of the location examined here. That research assumed a 75 percent reduction in capacity. The study concluded that for a 60-min incident at the southern location, the impact on the system amounted to slightly more than 16,000 vehicle-hr of delay, or roughly the same as the incident simulated here lasting approximately twice as long. The Mannering study also indicated that the longer the incident is in place, the greater is the relative impact of not clearing the incident. For example, the impact between clearing the incident in 60 min versus 50 min was estimated to be about 5,400 vehicle-hr of delay. The results of the Mannering study lend credence to the magnitude of impact presented in this paper.

The final step of determining the benefit-cost relationship of using total stations as an incident management tool was to assign costs to the benefits and compare these costs to the purchase price of the equipment. The field equipment cost WSDOT about $15,000/system. The estimated benefit of 7,000 vehicle-hr of delay savings and an assumed value of time of $7.00/vehicle-hr produce a benefit in reduced delay of $49,000. For a single peak-period accident, the benefit is more than three times the cost of the equipment. Given that the value of time is a very controversial subject, the researchers determined the break-even point for the investment on the basis of reduced delay. The equipment would have paid for itself on the single accident simulated if the value of time was as low as $2.14/hr.

The research team also estimated the benefit in reduced fuel consumption. Lindley's work suggests that each vehicle hour of delay wastes about 1.1 gal of fuel (*1*). Therefore, the estimated fuel savings for the simulated accident are 7,700 gal because of the use of total stations. Assuming a very conservative estimate of $1.00/gal of fuel, the use of total station equipment to investigate the accident simulated saved $7,700 by reducing fuel consumption. On savings in fuel consumption alone, the price of the equipment would have been offset with the second peak-period accident investigated.

Although this benefit-cost analysis was relatively simplistic, using many assumptions, the magnitude of the traffic benefits clearly outweigh the cost of the equipment.

## CONCLUSIONS

### Advantages of Total Station Investigation

The most dramatic advantage of using total stations for accident investigation is the reduced delay to the traveling public. The data from urban freeway accidents investigated with total stations and the coordinate method indicate an average clearance time savings of 51 min per investigated accident. Regardless of any assumptions concerning the cost of fuel and value of time, the equipment cost is offset on the first peak-period accident.

Because the equipment can measure over the tops of cars, respondents rarely have to close the roadway completely, if at all. If the road is closed, the closure is shorter because the total station measurement is much faster than previous methods. These differences affect the occurrence and severity of congestion and subsequent secondary accidents, user costs, excess fuel consumption, and driver frustration.

The use of total stations also has some advantage to respondents. First, it increases safety for the investigators. Because scene details can usually be measured from one site and the total stations can measure over the tops of vehicles, investigators have much less need to be in the roadway. Measuring is also faster. As seen in the preceding data, on-scene investigation with total stations takes less than half the time to investigate that the coordinate method requires. Therefore, investigators are in or near the roadway, exposed to traffic, for much less time.

The equipment is also small and portable. It easily fits into a car trunk already filled with police and investigative equipment.

Also beneficial are the computerized drawings, which can be produced faster and can convey more information than the hand drawings. Washington State Patrol has found the computerized data base of reusable site drawings especially useful for complex intersections.

In addition to operational benefits, the total station method is important in tort liability cases. The equipment produces a more detailed accident description and a more professional drawing of the scene. The system's ability to work with a microcomputer to animate the accident scene gives a jury a better understanding of the events, causes, and effects of an accident. Better records are also retained in the files. Overall, total station accident investigation removes much of the ambiguity of accident recreation that is common when a trial does not take place until several years after an accident has occurred.

In Washington State, this benefit is important enough that a major accident investigation team has been formed. The team comprises two commissioned Washington State Patrol officers (one responsible for collision investigation and one for interviewing witnesses); one Washington State Patrol commercial vehicle enforcement officer to assess vehicle condition; and one WSDOT engineer to record and document the location and condition of the roadway, traffic control devices, and roadway safety appurtenances. The total station is the central piece of equipment that the investigation team uses in its investigations.

Perhaps the best indication of the benefits of the total station method for accident investigation is that the Washington State Patrol has bought three additional systems and anticipates buying two more. With this additional equipment, every investigation team in the patrol will be equipped with a total station system.

### Potential Disadvantages of Total Station Investigation

Like any technology does, the total stations have a few disadvantages. The first is their cost. Costs vary somewhat by manufacturer, but their prices range from $15,000 to $20,000, depending on the office equipment needed. However, as is shown in the benefit-cost analysis, the equipment cost can be recouped in the first peak-period accident investigation.

The system also has some minor physical limitations. It is hard to use in dense fog because it is difficult for the investigators to site the prism. Measurement can also be affected by heat waves on extremely hot days. The stations run on rechargeable batteries, so investigators must keep extra batteries ready. Finally, although the systems are fairly easy to use, training is necessary, and investigators should use the systems often to remain proficient.

## RECOMMENDATIONS

The information presented shows that electronic total stations can be used to reduce incident clearance times in a part of the incident management process that is seldom viewed as subject to time reduction: accident investigation by enforcement personnel. The researchers highly recommend the use of this tool as part of an overall incident management system. The clear benefits to traffic management during incidents indicates that highway agencies and departments of transportation are the agencies receiving most of the benefit of these systems. The researchers recommend that these agencies consider the use of total stations as an element of their incident management systems to be funded in the same manner as other incident management strategies.

## ACKNOWLEDGMENTS

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# Effect of Intersection Congestion on Accident Rates

J. W. HALL AND MARGARITA POLANCO DE HURTADO

Although there is general agreement that accident rates are highest during the periods of the day when traffic volumes are low, the change in accident rates as facilities become more congested is poorly understood. The peak-hour accident rates at several hundred signalized intersections in Albuquerque were examined in relation to their level of congestion, as reflected by volume/capacity (*v/c*) ratios. Accident rates exhibited a small but significant positive correlation with the amount of entering traffic. On average, intersection accident rates are at a minimum in the range $0.6 \leq v/c \leq 0.8$. However, equations developed for estimating accident rates as a function of *v/c* had such high standard errors that they could not be used for predictive purposes.

Balancing the competing demands for roadway improvements is a difficult task. As peak-period traffic flows continue to increase, engineers endeavor to improve roadway operation by reducing the extent and duration of congestion while trying to increase safety for all road users. The sets of treatments that can produce smooth traffic flow or improved highway safety are reasonably well established. Operating under financial constraints, however, a traffic engineer may be forced to choose between an expenditure to enhance safety and one to upgrade highway traffic flow. Certain traffic flow improvements may also enhance safety, but little is known about the interaction between safety and traffic flow treatments for roadway sections or spots. This project was undertaken to determine the existence and nature of any relationship between the degree of congestion and the level of safety at urban signalized intersections.

## BACKGROUND

The relationship between traffic flow and accident experience is poorly understood, especially under conditions in which traffic flow approaches capacity. There is convincing evidence that accident rates are highest at night, when traffic flows are relatively low. This may be due to sleepy or impaired drivers, the difficulties of driving in the darkness, or the small denominator in the accident rate equation. It is clear, however, that most capacity-type improvements will not be of significant benefit during low-volume nighttime conditions.

The technical literature provides little guidance on what to expect near the other boundary condition, at which the ratio of traffic volume to capacity (*v/c*) approaches 1. Higher traffic volumes and the larger denominator in the accident rate equation would tend to decrease the accident rate. However, the stop-and-go traffic conditions that prevail as facilities become congested provide many opportunities for collisions. In these situations, improvements to facility capacity would decrease the incidence of stop-and-go driving, but their effect on accident experience is less obvious. One might conclude that a facility with better traffic flow must inherently provide safer operation. Indeed, the 1965 *Highway Capacity Manual* (HCM) stated that level of service (LOS) is "a qualitative measure of the effect of a number of factors, which include speed and travel time, traffic interruptions, freedom to maneuver, safety, driving comfort and convenience, and operating costs" (*1*). However, the manual failed to provide convincing support for this statement.

Realistically, there may be trade-offs between factors that improve traffic flow and those that improve traffic safety. One example comes from the increasingly popular treatments to enhance traffic flow on urban freeways. There is evidence that, other factors being equal, travel lanes 12 ft wide are safer than narrower lanes. It is also clear that a roadway with three 12-ft travel lanes and wide shoulders can be restriped to provide four 11-ft lanes and narrower shoulders. Conventional wisdom suggests that such a change would enhance capacity at the expense of safety. Using procedures of the 1985 HCM, the calculated capacity increases by approximately 28 percent (*2*). However, evaluations of 11 installations of this type found that the accident rate was *reduced* by 33 percent (*3*). Apparently, the reduced stop-and-go driving brought about by the capacity improvement more than counterbalanced the potential detriment to safety associated with narrower lanes. Of course, the real world rarely provides opportunities to evaluate the preferred alternative of changing from three congested 12-ft lanes to four free-flowing 12-ft lanes.

A previous study in New Mexico attempted to identify a relationship between traffic flow and accident rates on rural roads (*4*). The study found that the highest accident rates occurred during the low-volume hours between midnight and 6 a.m. Accident rates on these roads tended to decrease during the higher-volume daytime hours. Traffic volumes at the study sites rarely approached *v/c* ratios as high as 0.2, so it was not possible to examine highway safety under congested traffic flow conditions. However, a pilot study of four urban intersections, undertaken as part of this project, suggested that accident rates begin to increase at higher *v/c* ratios.

## STUDY PROCEDURE

The current study was undertaken to evaluate the variation in accident rates that accompanies changes in the degree of

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congestion, as reflected by the $v/c$ ratio. To examine the traffic flow–safety relationship over a complete range of $v/c$ ratios, this study was restricted to urban intersections. The initial study plan relied on secondary sources to assemble peak-period traffic volume and accident data for signalized intersections in Albuquerque. With supplementary data necessary to calculate intersection capacity, these data would be used to determine capacity, $v/c$ ratios, and accident rates. It quickly became apparent that the city of Albuquerque had relatively few recent traffic volume counts that would meet the needs of this study. In fall 1989, however, the city initiated a contract with a consultant to collect peak-period traffic volumes and capacity information at approximately 445 signalized intersections; these data were made available to this project.

An accident analysis for Bernalillo County, which includes Albuquerque, was undertaken to assess the variation in crash experience by hour of day and to determine other crash characteristics of general interest. During 1987–1989, there were 65,500 reported traffic accidents in the county; 98 percent of these were in Albuquerque. The morning and evening peak traffic periods (7:00 to 9:00 a.m. and 4:00 to 6:00 p.m.) accounted for 28 percent of all accidents. In Albuquerque, 36 percent of the crashes occurred at intersections and an additional 25 percent were described as intersection-related (occurring within 200 ft of an intersection).

The data bases that emerged from the collective efforts of the researchers and the city's consultant consisted of the following:

1. A listing of 1987–1989 Albuquerque intersection and intersection-related traffic accidents during weekday peak periods; the data included single- and multiple-vehicle accidents and pedestrian accidents. The data base was subsequently restricted to crashes at signalized intersections during the morning and evening peak hours.
2. The 1990 weekday peak-hour intersection volumes for all of these intersections.
3. The morning and evening peak-hour $v/c$ ratios for each signalized intersection, calculated using the procedures of the 1985 HCM and the design and operational characteristics of the individual intersections (*2*).
4. Morning and evening peak-hour accident rates for each intersection, using the peak-hour accident and traffic volume data just described.

A preliminary screening of the traffic volume, $v/c$, and accident data bases identified several problems that led to the exclusion of certain observations from the subsequent analysis. Several traffic signals, primarily near shopping centers, operate in a flashing mode during the morning peak and in a normal mode during the afternoon peak; to make the samples comparable, these intersections were excluded. Approximately 40 intersections had traffic signals installed between 1987 and 1989, but because the volume data collected in 1989–1990 may not reflect the conditions at these intersections over the analysis period, these intersections were also dropped from consideration. Finally, intersections of city streets and freeway on- and off-ramps were excluded; volume data for these locations appear to be reliable, but experience has shown that computerized accident location information for these sites is often incorrect. These exclusions reduced the sample size to 326 signalized intersections.

## CHARACTERISTICS OF DATA BASES

The accident record file does not identify all signalized intersections, so it was necessary to match the traffic volume file with the intersection accident data base manually to determine the number of weekday accidents occurring at each location during its morning and evening peak hours. The resultant information (consisting of the entering volumes, the estimated entering traffic for the 3-year study period, the number of accidents, and the $v/c$ ratios) was summarized on separate spreadsheets for the morning and evening peak hours.

The signalized intersections considered encompass a wide variety of design and operational characteristics. Speed limits on the approaches vary from 25 to 50 mph. The approaches themselves range from single lanes shared by through, left-turn, and right-turn traffic to those with multiple through lanes, dual left-turn lanes, and exclusive right-turn lanes. In accord with the traffic volume counting techniques used by the consultant, the 60-min peak hours could begin at any 15-min interval between 7:00 and 8:00 in the morning and 4:00 and 5:00 in the afternoon. The most common peak-hour beginning times were 7:15 and 7:30 a.m. (each accounting for 40 percent of the morning peak hours) and 4:30 p.m. (half of the afternoon peak hours). Table 1 shows the average and range of values for the peak-hour entering volume on all approaches, the peak-hour accidents, and the accident rate per million entering vehicles (mev).

A surprising number of sites had no peak-hour traffic accidents in or within 200 ft of the signalized intersection during the entire study period. Of the 326 intersections, 60 (18 percent) had no accidents during the morning peak hour and 45 (14 percent) had no accidents during the evening peak hour. Overall, the sites averaged 2.7 accidents during the morning peak hour and 3.9 accidents during the evening peak hour. The signalized intersection accidents in this sample account for about 27 percent of Albuquerque's non-Interstate accidents during the morning and evening peak hours.

## PRELIMINARY ANALYSES

The initial step in the analysis compared an intersection's peak-hour crash frequency with its entering volume during the same hour. Both the number of accidents and entering traffic volume are discrete variables, but volume, which ranges

**TABLE 1 Peak-Hour Characteristics of 326 Study Intersections**

| | Morning | Evening |
|---|---|---|
| Entering Volume (vph) | | |
| Lowest | 209 | 309 |
| Average | 2240 | 2840 |
| Highest | 5877 | 7474 |
| Accidents | | |
| Lowest | 0 | 0 |
| Average | 2.7 | 3.9 |
| Highest | 14 | 27 |
| Total Accidents | 890 | 1274 |
| Accident Rate (per mev) | | |
| Lowest | 0.00 | 0.00 |
| Average | 1.56 | 1.76 |
| Highest | 5.85 | 14.41 |

from 200 to 7,500 vehicles per hour, effectively behaves like a continuous variable. Figure 1 plots the 3-year accident frequency as a function of entering traffic volumes for the morning peak hour. The plot clearly demonstrates the discrete nature of the accident frequency in comparison with the continuous nature of the volume variable. The figure includes a least-squares regression line relating entering traffic volume to accident frequency. This line was forced to go through the origin because the calculated intercept value was not significantly different from zero. Predictably, this line shows that crash frequency increases with higher entering traffic volumes; the plot for the evening peak hour is similar. The equations relating accident frequency ($A$) to peak-hour volume ($V$) are as follows:

Morning:

$$A = +1.26 * V/1{,}000 \tag{1}$$

Evening:

$$A = +1.44 * V/1{,}000 \tag{2}$$

The $r^2$-value for Equation 1 is .43, indicating that 43 percent of the observed variation in the number of morning peak-hour accidents is explained (but not necessarily caused) by changes in the entering traffic volume. The corresponding coefficient for the evening peak hour is .38. Both coefficients are statistically significant. Equations 1 and 2 suggest that, on average, the number of peak-hour accidents during a 3-year period will increase by 1.3 to 1.4 for each additional 1,000 vehicles per hour of entering traffic. Although these results are in reasonable agreement with intuition, and the equations provide an indicator of the expected accident changes associated with increases in volume, the scatter exhibited in Figure 1 limits the credibility of such a prediction.

It is potentially more meaningful to examine the variation in signalized intersection accident rates with increases in traffic volume. With the data available to this study, accident rates were calculated on the basis of the number of mev during weekday peak hours over the 3-year study period. As shown in Table 1, peak-hour rates at individual intersections ranged from 0.0 to 5.85 accidents per mev (morning) and 14.41 accidents per mev (evening). Average peak-hour accident rates were 1.56 and 1.76 accidents per mev for the morning and evening, respectively. Although intersection volume data for non–peak hours are not available for comparison, a coarse analysis of Bernalillo County's travel-based accident rate variations with time of day shows that rates during other daytime hours are somewhat lower.

The distribution of morning peak-hour entering traffic volumes and intersection accident rates is presented in Figure 2. Because of the discrete nature of the number of accidents, $n$, many of the individual data points appear to fall on a family of curves. Assuming 260 weekdays for each of the 3 years in the study period and a peak-hour entering volume of $V$, the weekday peak-hour accident rate ($R$) is given by

$$R = (n * 1{,}000{,}000)/(3 * 260 * V) = 1{,}282.05 * n/V \tag{3}$$

For $n = 3$, the accident rate as a function of the entering traffic is given by

$$R = 3{,}846.15/V \tag{4}$$

The relationship in Equation 4 is plotted as a dotted line in Figure 2. The corresponding curves for values of $n \leq 9$ are evident from the data points in the figure. The dashed line toward the bottom of Figure 2 represents the least-squares regression; its small slope (0.00012) is statistically significant ($t = 2.05$). In other words, intersection peak-hour accident rates and the volume of entering traffic are not independent; instead, they have a small positive correlation. The relationship for the evening peak hour is quite similar; the slope of the regression line is 0.00015, and it is significantly different from zero ($t = 2.71$). Statistically, therefore, accident rates do increase as the volume of traffic entering the intersection increases. However, the practical significance is questionable, since the peak-hour intersection accident rate would increase by only 0.12 to 0.15 for every additional 1,000 entering vehicles.

A primary conclusion from this analysis is that little of the variation in intersection peak-hour accident rates can be explained by changes in the amount of entering traffic. Considering the wide differences in the design and operating characteristics of urban intersections, this is not particularly surprising. Nevertheless, it is appropriate to consider other

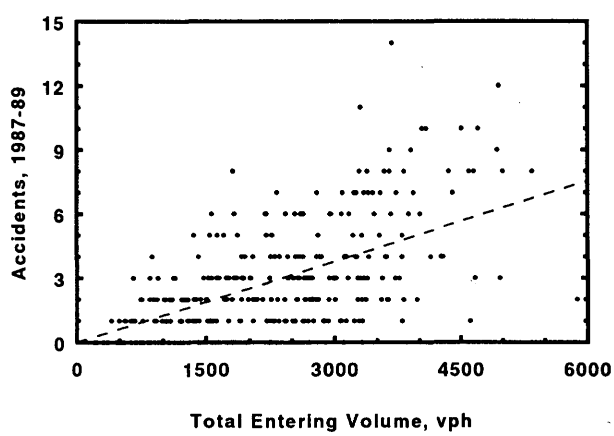


**FIGURE 1 Morning peak-hour accidents.**

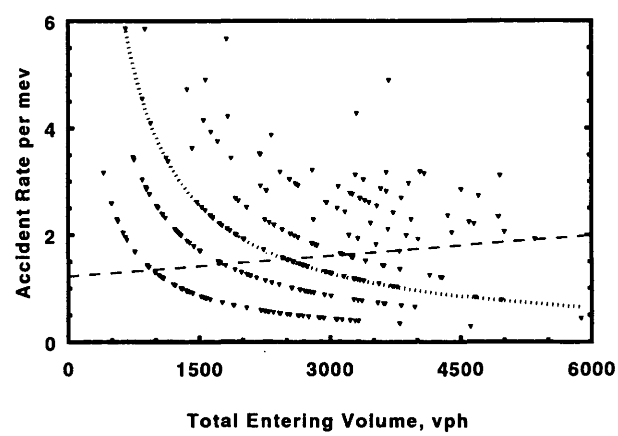


**FIGURE 2 Morning peak-hour accident rates.**

factors that may account for this variation. As discussed earlier, one factor that may help explain different levels of accident experience at intersections is the amount of intersection congestion, which is reflected by the *v/c* ratio.

## INTERSECTION CAPACITY ANALYSIS

The 1985 HCM identifies many variables that can influence the quality of intersection operation, including signal timing, lane width, approach grades, vehicle types, and demand for turning traffic movements. As part of its contract with the city of Albuquerque, the consultant assembled the relevant data at signalized intersections. Certain pieces of information, such as traffic volume, were collected in the field for each intersection. Geometric data were gathered from existing city files, and other data of lesser importance (e.g., percentage of heavy vehicles in the traffic stream) were collected using sampling techniques. The actual analyses were performed using the Highway Capacity Software, a widely used set of computer programs that replicate the analysis procedures of the 1985 HCM.

The intersection of Tramway Boulevard and Manitoba Street in northeast Albuquerque serves as a straightforward example to highlight *v/c* analysis and the effects of traffic growth. This four-leg intersection does not have significant problems with accidents or capacity; however, its relatively simple design facilitates a description of capacity analysis. The intersection has an exclusive left-turn lane and a single through/right-turn lane for north- and southbound traffic on Tramway. The east- and westbound approaches on Manitoba also have two lanes. When the intersection was counted in December 1989, the total entering volume during the evening peak hour was 1,794 vehicles, more than half of which were traveling northbound on Tramway.

The Highway Capacity Software adjusts the individual approach volumes on the basis of such factors as the grade of the approach roadway, the peak-hour factor, and the traffic platoon arrival type; it subsequently calculates measures of performance by lane, lane group, and approach. In 1989 this intersection was operating at LOS A during the evening peak hour, with an average delay of 3.9 sec vehicle and a *v/c* ratio of 0.614. These operating parameters would be considered quite good. As land use in the vicinity continues to develop, however, the peak-hour traffic volume using the Tramway-Manitoba intersection will increase and operating conditions will probably deteriorate. It is difficult to predict precisely how fast the traffic will grow or how the growth will be distributed among the various intersection approaches and movements. For illustrative purposes, the intersection geometrics and signal timing were held constant while the traffic volumes for all movements were increased in 5 percent increments. The capacity analyses were redone for each higher volume level up to a 63 percent increase (total entering volume of 2,925 vehicles per hour), when the intersection was operating at capacity ($v/c = 1.0$ and delay = 65.2 sec vehicle). With progressive increases in volume, the *v/c* ratio increased in a linear manner. On the other hand, the average delay per vehicle exhibited only a moderate increase for volumes up to 30 percent higher than current values; beyond this point, delays increased substantially. The projected changes in *v/c* ratio and delay with increases in entering volume are shown in Figure 3. The dotted lines in the figure represent the limits for LOS ranges.

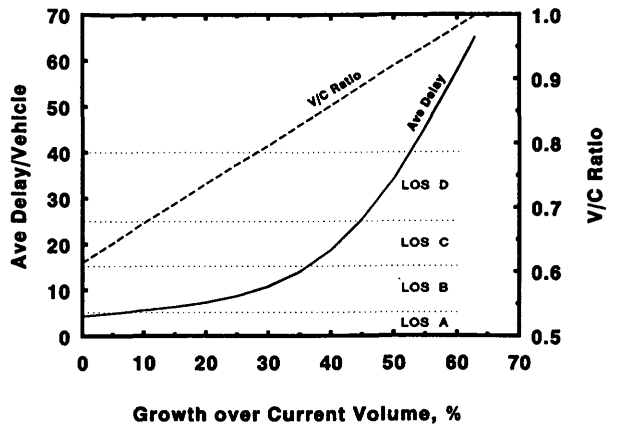


**FIGURE 3 Effects of volume increases.**

The deterioration in intersection operation at higher volumes is not surprising. Similar results could be expected at more complicated intersections with more lanes and more elaborate signal timing. It is less obvious, however, that according to the techniques of the 1985 HCM, intersection operation can deteriorate significantly with increasing volumes on just a single approach. At the Tramway-Manitoba intersection, for example, if only the northbound volumes are increased by 63 percent while volumes on the other three approaches are held steady, the total volume entering the intersection will increase by 33 percent. If the geometrics and signal timing are held constant, the *v/c* ratio will increase to 0.983 while the average delay will be 64.1 sec—results very close to those obtained for the previous example, even though the total volume is substantially less. This example demonstrates high demand on a single approach can be sufficient to produce a high *v/c* ratio for the entire intersection. In addition, relatively minor changes in signal timing to improve the operation of a high-volume approach (while adversely affecting the other, lower-volume approaches) can significantly reduce the *v/c* ratio and delay for the entire intersection. The sensitivity of *v/c* ratios to these parameters should be kept in mind when interpreting the subsequent analyses of accident rates and congestion.

## OPERATION AT HIGH *v/c* RATIOS

In theory, the actual volume entering an intersection should be less than the intersection's capacity. In practice, intersections at which the entering volume exceeds the calculated capacity are routine. These results could be due to one or both of the following factors:

1. Errors in collecting processing data; relatively small data discrepancies can result in sizable changes in the calculated capacity.
2. The inability of the techniques of the 1985 HCM to account properly for all of the parameters that may influence intersection capacity.

Although it is outside this project's scope to resolve either of these potential problems, the fact that peak-hour *v/c* ratios

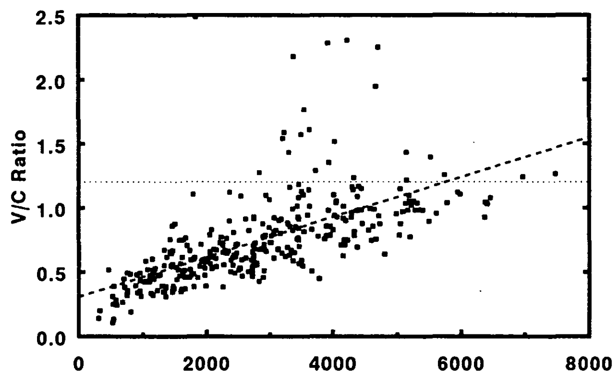


**FIGURE 4 Entering volume versus *v/c*.**

exceed unity at 10 percent of the Albuquerque study intersections during the morning and at 18 percent of them during the evening suggests that this situation is fairly common. The 1985 HCM indicates that some of its analysis procedures may not produce reliable results when the *v/c* ratio exceeds 1.2.

Before proceeding, it is appropriate to consider how the *v/c* ratio relates to the actual volume of traffic entering an intersection. Ideally, intersections should be designed to accommodate their peak-hour volumes; those with relatively lower demand volumes would have smaller capacities, and those with larger demands would be built and operated to provide greater capacities. Within limits of good design practice, high-volume intersections might use multiple approach lanes, exclusive right-turn lanes, or dual left-turn lanes. If this ideal were achieved, all signalized intersections would provide similar levels of service during peak hours and their *v/c* ratios would be independent of the entering traffic volume. A review of the data, however, found a strong positive correlation between peak-hour entering traffic volumes and *v/c* ratios. Figure 4 shows the distribution of these two parameters during the evening peak hour. The dashed least-squares regression line in the figure has an $r^2$ of .40, indicating a highly significant relationship between peak-hour volume and *v/c* ratio. In practical terms, this implies that capacity deficiencies rarely exist at moderate-volume intersections but that they are routine at high-volume intersections. Unfortunately, many of the busiest intersections have already been modified to enhance capacity.

Figure 4 also demonstrates the spread of evening peak-hour *v/c* ratios, with values ranging from 0.11 to 2.49. There are 23 intersections with *v/c* ratios greater than 1.20, including five with ratios in excess of 2.0. To avoid the bias that these outliers may introduce into subsequent analyses, and in compliance with the general admonitions in the 1985 HCM, intersections with peak-hour *v/c* ratios greater than 1.2 were deleted from further consideration. The remaining intersections have average *v/c* ratios of 0.60 during the morning peak hour and 0.68 during the evening peak hour.

## ACCIDENT RATES AND *v/c* RATIOS

The intersections remaining in the data bases after these deletions had a mean rate of 1.54 accidents per mev and a *v/c* ratio of 0.60 in the morning; the comparable values for the evening were 1.79 and 0.68, respectively. These data support the conclusion that congestion does affect accident rates. Specifically, average accident rates are 16 percent higher in the evening peak hour than in the morning and average *v/c* ratios are 13 percent higher. Because these comparisons involve almost identical sets of intersections, it is tempting to attribute the higher evening accident rates to an increase in congestion. In actuality, however, the difference may be due to other differences, such as changes in human behavior characteristics, between the two peak periods.

The data bases described earlier were used in an effort to identify the existence of a relationship between peak-hour accident rates and intersection congestion. A plot of these variables for the evening peak hours is presented in Figure 5. The scattered data points do not suggest any obvious functional relationship between *v/c* ratios and intersection accident rates. The least-squares linear regression line has a positive slope, but the $r^2$-value is only .01. Application of least-squares techniques to determine the best quadratic relationship between these parameters yielded a line nearly identical to the linear relationship. Results during the morning peak hours were equally disappointing. These findings indicate that changes in the *v/c* ratio over the range 0 to 1.2 explain a negligible amount of the variation in accident rates.

The difficulty in establishing a functional relationship between *v/c* ratios and accident rates may be partly due to the inclusion of individual intersections that pose absolutely no problem from the perspective of either safety or capacity. Since one of the practical objectives of this research was to help the engineer faced with the dilemma of devoting scarce resources to the improvement of either safety or capacity, it seems reasonable to focus on intersections that may be deficient in either or both of these areas. It is unlikely, for example, that an engineer would undertake safety improvements for an intersection that had no peak-hour accidents during the previous 3-year period or capacity improvements at an intersection operating at LOS A. Near the other extreme, intersections with 10 peak-hour accidents or ones operating at LOS D would be good candidates for further study and possible treatment.

For peak-hour conditions in Albuquerque, the dividing line between safe and unsafe, or between uncongested and congested, is not obvious. The inherent reliability problems of accident-, volume-, and capacity-related data need to be rec-

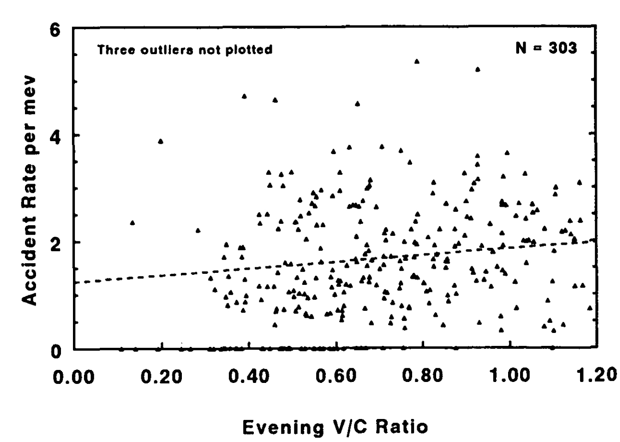


**FIGURE 5 Evening *v/c* ratio versus accident rate.**

ognized in making such a decision. High thresholds could be used to identify unsafe or congested intersections, but they would overlook sites that are approaching these undesirable conditions. Furthermore, the use of high accident rates or $v/c$ ratios to distinguish good from bad operation would ignore many potentially troublesome locations. To avoid these problems, this project selected threshold values below which there should be universal agreement that the intersection is both safe and uncongested. Specifically, the modified sample was limited to those intersections with more than one peak-hour accident in the past 3 years *or* with a peak-hour $v/c$ ratio greater than 0.5, roughly corresponding to the upper limit of LOS A.

These revised data bases included 242 intersections during the morning peak hour and 254 during the evening peak hour. Attempts to predict accident rates as a function of $v/c$ ratio using linear regression techniques produced encouraging results, but the $r^2$-values were only $\approx .03$. Preliminary findings from an earlier study had indicated that the accident rates ($R$) were highest for low and high values of $v/c$ and were lower at intermediate values (*4*). This type of relationship can be modeled with a quadratic equation of the form

$$R = a + b * (v/c) + c * (v/c)^2 \quad (5)$$

Standard analysis techniques were used to find the best-fit quadratic equations for the morning and evening peak hours. The results were moderately successful; $r^2$-values were .12 and .09 in the morning and evening, respectively. The plot of evening accident rates as a function of $v/c$ ratios is shown in Figure 6, along with the best-fit quadratic equation indicated by the dashed line. The corresponding morning plot is similar: both quadratic curves indicate a minimum accident rate in the vicinity of a $v/c$ ratio of 0.8, with greater accident rates predicted for both higher and lower values of $v/c$.

Although the relationships indicated by the quadratic equations are statistically significant, they may have limited practical significance when applied to a particular location. Consider, for example, an intersection with an evening peak-hour $v/c$ ratio of 0.8. Figure 6 shows that existing intersections with $v/c$ ratios of $0.80 \pm 0.03$ have accident rates ranging from 0.40 to 3.14 per mev. For a $v/c$ ratio of 0.8, the model predicts

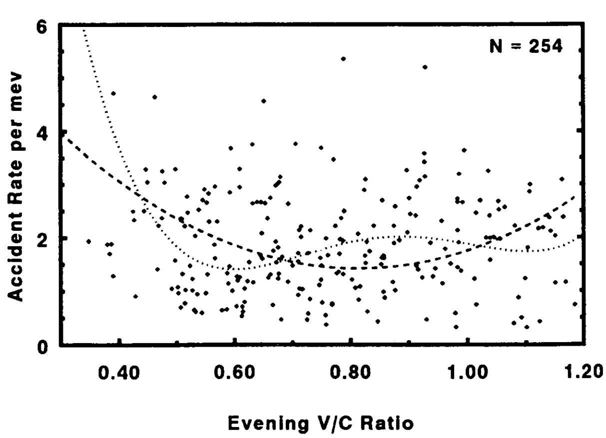


**FIGURE 6 Sites with accidents or $v/c$ greater than 0.5.**

a rate of 1.33 accidents per mev. An analyst examining a particular intersection should, of course, be able to determine the current values of $v/c$ and $R$ at the location. However, the model provides limited guidance for estimating the accident rate if the $v/c$ ratio is projected to increase in the future. For example, if $v/c$ is predicted to grow from 0.8 to 1.0, the model indicates that, on the average, the accident rate will increase from 1.33 to 1.52 per mev. Unfortunately, the 90 percent confidence interval associated with this prediction is $1.52 \pm 1.78$, or $-0.26$ to 3.30 per mev. Thus, whereas the model supports the premise that the anticipated increase in the $v/c$ ratio in this particular range will be accompanied by a higher accident rate, it is clearly not precise enough to use to predict future accident rates.

Given the wide scatter of the data, it is difficult to imagine that a superior (and realistic) model could be developed to describe accident rates as a function of $v/c$ ratios. Nevertheless, from a mathematical perspective, the best-fit line can be improved by incorporating higher-order terms, such as $(v/c)^3$ and $(v/c)^4$, in an expanded version of Equation 5. One effect of adding these higher-order terms is to move the best-fit line "closer" to a greater number of data points, thus improving the $r^2$-values. Another effect is to increase the number of inflection points along the curve; not surprisingly, this results in a shift of the minimum accident rate to a lower value of $v/c$. For example, equations incorporating both $(v/c)^3$ and $(v/c)^4$ terms produce the dotted curve in Figure 6, with minimum accident rates in the vicinity of a $v/c$ ratio of 0.6. Without arguing the relative merits of models of the form of Equation 5 versus those containing higher-order terms, it appears realistic to assume that minimum intersection accident rates during peak hours are most likely to occur for operating conditions in the range $0.6 \leq v/c \leq 0.8$.

## CONCLUSIONS AND RECOMMENDATIONS

The research described in this report has attempted to develop a relationship between the level of congestion and the peak-hour accident rates at urban signalized intersections. Although this objective was not completely achieved, the project has developed some interesting and potentially useful information related to peak-hour accident experience. The relevant findings apply to peak-hour conditions in Albuquerque but should not be indiscriminately applied to other situations; they include the following:

1. More than 60 percent of the accidents in Albuquerque occur at intersections or are intersection-related.
2. The 60-min periods beginning at 7:15 and 7:30 a.m. each account for 40 percent of the morning peak hours at signalized intersections, and the hour beginning at 4:30 p.m. accounts for more than 50 percent of the evening peak hours.
3. More than a quarter of the peak-hour accidents occur at signalized intersections.
4. Accident rates at signalized intersections average 1.56 and 1.76 per mev during the morning and evening peak hours, respectively.
5. The number of peak-hour accidents at signalized intersections is highly correlated with the number of entering ve-

hicles; the peak-hour accident rate is weakly, but significantly, correlated with the number of entering vehicles.

6. Peak-hour *v/c* ratios vary widely among intersections; these ratios have a significant positive correlation with the volume of entering traffic.

7. Quadratic models explain approximately 10 percent of the observed variation in intersection peak-hour accident rates as a function of *v/c* ratios. However, the models are not sufficiently reliable to serve as predictive tools.

8. Minimum peak-hour accident rates tend to occur within the range $0.6 \leq v/c \leq 0.8$; higher *v/c* ratios tend to be associated with increasing accident rates.

There are several possibilities for extending the work initiated in this project. The project started with 445 signalized intersections, but more than 40 percent of them were dropped from the final analysis for the reasons cited earlier. With additional effort, including the manual review of hard copy accident reports, it might be possible to include some of the deleted intersections on interchange ramps and frontage roads. With the passing of time, it will be possible to include intersections that were signalized during recent years.

The analyses discussed in this report were based on accident information for 1987–1989 and traffic volume information for 1989–1990. Albuquerque plans to update its peak-period traffic volume data base annually by performing additional counts at a sample of intersections. If this is done, it will provide new volume and *v/c* data that could be used to monitor historical trends in the *v/c*–accident rate relationship at particular intersections. Although the sample sizes will be smaller than those available for this study, a detailed examination over time at specific intersections could avoid the influence of other factors, not studied in this project, that may contribute to crash occurrence.

## ACKNOWLEDGMENTS

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# Identification of Dangerous Highway Locations: Results of Community Health Department Study in Quebec

BRUCE BROWN, CÉLINE FARLEY, AND MICHELINE FORGUES

Dangerous highway locations on numbered highways within the territory of one community health department are identified. Three sources of information are used: police accident reports, a systematic inspection of all numbered highways, and a community survey of municipalities, police, and health service providers. The location of all police-reported fatal and serious-injury accidents were reviewed and corrected, and corrections were submitted to the reporting jurisdiction; this resulted in a 20 percent increase in the number of these reports attributed to numbered highways. The initial police-reported data included 11,538 accidents with and without victims occurring on the 271 km of numbered highways in the territory between 1984 and 1987. A weighting system based on the severity of injury for each police-reported injury was used in the initial screening process; the influence of differing weighting schedules using corrected and uncorrected location data is presented in a matrix. Weighted injury frequencies per unit distance and weighted injury rates per 100 million vehicle-km are presented for all sites and for all numbered highway segments. Priority sites are ranked considering injury frequencies and injury rates. The convergence of identification by police-reported data, by highway inventory, and by community reporting is presented. The 28 priority sites retained for further study cover about 6 percent of the numbered highways in the territory but account for 53 percent of deaths, 30 percent of serious injuries, and 32 percent of minor injuries from accidents reported by police.

Community health departments (DSCs) in Quebec have been working in the field of highway safety since the early 1980s. At the Sixth Canadian Multidisciplinary Road Safety Conference, the departments in the Monteregie area of Quebec presented an overview of their work on identifying dangerous highway locations in their region (*1*). We now report the final results of this work in greater detail for the territory of one of the four participating departments.

The identification and correction of hazardous highway locations have received a great deal of attention across North America over the past 25 years; it is particularly evident in the engineering literature. Reviews of methodology for this work are available as well as recent reviews of the relationships between specific highway infrastructure elements and the frequency of roadway-associated accidents and the severity of associated injuries (*2–5*). Travel-lane width, shoulder width and surface condition, sideslope characteristics, and highway geometry, particularly the presence of curves, have been related to injury severity and frequency. Administrative regulations requiring the identification and correction of dangerous highway locations in the United States have been defined by federal law since 1966. Cost-effectiveness and cost-benefit analyses have been proposed to guide highway rehabilitation programs with respect to infrastructure elements (*6*). Methodologic issues have recently received more attention, particularly concerns about the need to correct for regression-to-the-mean phenomena when the evaluation of intervention effectiveness is done for "dangerous locations" (*7*).

In Quebec during the past 6 years, more than 10 studies identifying dangerous highway locations have been published by DSCs and university groups. With the exception of one study on bridge accidents, we know of none published in indexed peer-reviewed journals. The Ministry of Transport in Quebec has also been concerned by this subject and in 1990 announced a funding program for the correction of dangerous highway locations.

Inadequacies of the localization methods used in police reports in Quebec have been identified repeatedly. The coordinate localization is based on "mercators," 1- × 1-km squares defined by longitude and latitude numbers; this is the standard computerized localization method used by police in Quebec. In 1987 the Ministry of Transport announced its intention to introduce a link-node identification system for accident localization; this project has since been transformed into a project localizing sites using satellite-based technologies.

We present the methodology and results of our hazardous highway localization work developed from a public health perspective applied at a local level. We view this method as a screening tool for the presumptive identification of unrecognized (or at least uncorrected) dangerous highway locations (*8*). In much the same sense as medical screening tests, the "cases" being identified (in this discussion, dangerous highway locations) need further investigation before the initial diagnosis is confirmed or rejected. As in all screening tests, some cases will be identified falsely as being dangerous sites (false positives) and some dangerous sites will not be identified (false negatives). We must emphasize that a screening test is not diagnostic and represents only an initial examination that must be followed up by more investigation. The evaluation of the ability of a screening test to discriminate between cases and noncases is dependent on a "gold standard" that identifies cases with and without the condition being studied. The prevalence of the condition in the population will influence the predictive value of the test in the study population.

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In addition, the usefulness of a test will depend on a series of defined factors that include the importance of the condition (in terms of costs and suffering), the availability of effective treatment for identified cases, and some knowledge that the early identification (before the condition becomes symptomatic) of the condition will benefit the patient or society. The analogies for the case of dangerous highway locations are clear, but the absence of a true standard against which presumptive cases are judged remains a practical problem. We have dealt with this by choosing a somewhat arbitrary cutoff point, selecting only the "most extreme" cases in our population.

## STUDY AREA

The geographical area examined by this study is on the south shore of the St. Lawrence River near Montreal and covers an area of about 100 × 20 km; 438,000 people live in the area. It is part of a larger administrative territory, the Monteregie, for which we present police-reported motor vehicle mortality data in Table 1.

In Table 1 we compare estimates of rates of death per 100 million vehicle-km of travel on our region's highways with those on U.S. highways. The U.S. data include 100 percent of motor vehicle–related deaths occurring in the United States as reported by the Fatal Accident Reporting System. We would like to underline two points:

1. The death rate increases with decreasing infrastructure quality for the numbered highways.
2. Most deaths associated with roadway use occur on numbered highways. In the case of the U.S. data, 57 percent of deaths occur on 22 percent of the total roadway distance.

Interpreting Table 1 to indicate that highways in the Monteregie are "worse" than those in the United States because death rates for each roadway category are higher in the Monteregie should be done with caution. It should be noted, however, that when these data were collected, rates of seat belt use were about three times higher in Quebec than in the United States. If all other factors were equal, and if seat belt use does effectively reduce the likelihood of death, one should expect lower death rates on Monteregie roads. It would be quite simple and inexpensive to use the methodology presented in this paper to determine the quality of highway infrastructure in different jurisdictions.

On the basis of our interpretation of Table 1—using in part the logic attributed to Jesse James when asked why he robbed banks ("Because that's where the money is")—this study is limited to the identification of hazardous highway locations on numbered highways. The rest of the paper deals with the data on numbered highways given in Table 2.

Information about the 271 km of numbered highways was thus examined. In the 4 years of reporting, 11,538 accidents were reported on these highways, of which 2,232 were accidents with victims. These represent 29 percent of all reported accidents, 58 percent of all deaths, and 38 percent of all severely injured victims in our territory.

## METHODOLOGY

The definition of hazardous highway locations used in this study is that proposed by Zegeer: "highway spots, intersections or sections with an abnormally high accident experience (frequency, severity or rate) or potential" (*2*).

The operational definition included all of these elements, that is, frequency, severity, rate, and potential for injury. The first three elements were derived from police accident reports and highway traffic flow and distance data available for all highways in our area (*9,10*). Treatment of these data is further defined later in the paper. The fourth element, accident potential, is derived from the systematic visual inspection of the 271 km of numbered highways using a methodology based on a report by Zegeer and further described later in the paper (*3*).

In addition to the accident report and highway inventory methods, we addressed a community survey questionnaire to all municipalities, community clinics, ambulance services, and municipal and provincial police in our area. They were asked about their perceptions of the importance of dangerous highway locations and the identification of specific sites. This methodology is also further described later.

To the best of our knowledge, this is the only report of such a combined identification methodology. Innovative fea-

**TABLE 1 Motor Vehicle–Related Fatality Rates for United States (1985) and Monteregie (1984–1987)**

| Highway category | Total number of deaths | | Fatality rate per $10^8$ veh-km | |
|---|---|---|---|---|
| | USA | Monteregie | USA | Monteregie |
| . Interstate | 4 200 | 118 | 0,7 | 0,9 |
| . Principal | 14 200 | 235 | 1,7 | 2,7 |
| . Secondary | 6 500 | 130 | 2,5 | 3,8 |
| . Unnumbered | 18 900 | 334 | 1,6 | N/A |
| Total | 43 800 | 819 | | |

Source: TRB (4) and calculations by community health departments from MTQ and Quebec Automobile Insurance Society Data (SAAQ).

TABLE 2 Distribution of Motor Vehicle Injury Victims by Severity of Injury and Category of Highway Site of Injury, Territory of DSC Charles LeMoyne, 1984–1987

| Highway category | Highway length (km) | Number of victims | | |
|---|---|---|---|---|
| | | Deaths | Severe injury | Minor injury |
| . Interstate (# 0 - 99) | 109 | 35 | 113 | 857 |
| . Principal (#100 - 199) | 122 | 60 | 407 | 2 267 |
| . Secondary (#200 - 299) | 40 | 6 | 46 | 294 |
| Total | 271 | 101 (58%) | 566 (38%) | 3 418 (26%) |
| . Entire territory | Not available | 175 | 1 483 | 13 000 |

Source: Quebec Automobile Insurance Society, Ministry of Transport of Quebec.

tures include the use of corrected localization data for all fatal and severe-injury accidents, the use of a severity index including only injury accidents, and the integration of these methods for selection of priority sites for further evaluation.

## Accident Report–Based Methodology

### Data Collection

All police-reported injury accidents were initially examined using a computerized data base. The location of fatal and severe-injury accidents (1,359 accidents, 1,658 victims) were corrected for the highway number and for the mercator number. As shown in Table 3, this resulted in a 20 percent increase in the number of reports attributed to numbered highways (from 438 reports before correction to 526 reports after correction). This is primarily due to the use of highway names without the corresponding number in some reports, particularly for highway sections passing through highly urbanized areas. The number of accidents occurring on numbered highways with both highway number and mercator identified increased by 88 percent (from 267 to 503 after correction). Nineteen percent of all fatal and severe-injury accident reports were corrected; mercator numbers were corrected only for reports attributed to numbered highways.

After corrections, a computer printout of all corrected reports was submitted to the police department responsible for having completed the report. Twenty-one police departments were contacted, and they confirmed, with few exceptions, the appropriateness of our corrections.

### Data Treatment

As a screening tool developed from a public health perspective, we chose to use injury victims as our unit of analysis. Most engineering literature reports use accidents, sometimes stratified by severity, as the unit of study. It is our understanding that the difference in the two units will be most evident in the case of severe frontal collisions; our method will in general attribute greater importance to these collisions because, for a given accident severity, the frontal collision will generate more victims than, for example, a single-vehicle fixed-object collision of equivalent accident severity. In effect, a single severe-injury accident that generates three severely injured victims will be counted three times in our system but only once in a classic engineering study.

An injury severity index that permitted the use of a single numeric value to express the total cost of all injuries associated with a particular location was applied. The values chosen were related to the direct and indirect economic costs of injuries as determined by the Quebec Automobile Insurance Society (*11*).

Fatal injuries were relatively undervalued in this system relative to Quebec economic cost data and costs based on other methodologies (*11–14*). The values attributed for different injury severity are as follows:

TABLE 3 Corrections Made to Accident Reports for Fatal and Severe-Injury Accidents, Territory of DSC Charles LeMoyne, 1984–1987

| | Highway number present | | Highway number absent | | Total |
|---|---|---|---|---|---|
| | Mercator number | | Mercator number | | |
| | Present | Absent | Present | Absent | |
| Before correction | 267 | 171 | 235 | 692 | 1 365 |
| After correction | 503 | 23 | 227 | 606 | 1 359 |

- Fatal injury—100
- Severe injury—20
- Minor injury—3

The weighted injury frequency for a particular location is calculated as the sum of the number of victims of a given severity ($Ni$) multiplied by the corresponding severity ($Si$), repeated for each severity level:

$$\text{weighted frequency} = \sum_{i=1}^{3} Ni \cdot Si \quad (1)$$

The weighted frequency for highway sections (from 12 to 47 km long) for each of the 12 numbered highways in our territory were also calculated as weighted frequencies per kilometer of roadway. Weighted injury frequencies were calculated for each of the mercators through which numbered highways passed.

We examined the influence of using corrected and uncorrected highway location data as well as the significance of the choice of severity index. This was done by comparing the 50 highest weighted frequency mercators that would be selected by using each of four different weighting schemes and comparing before-correction data with after-correction data. These comparisons are presented as a correlation matrix in Table 4.

The correction of location data alone resulted in a minimum of 9 (18 percent) and a maximum of 22 (44 percent) of the 50 mercators' changing. In our complete report we have also shown that between 10 and 34 percent of the 50 highest-frequency mercators change solely on the basis of the use of different weighted injury frequency scales (i.e., different severity indexes) (*15*).

The process of identifying individual hazardous sites using accident reports was done in two stages. The first stage involved selecting mercators with both high weighted injury rates and frequencies. In the second stage, data for these mercators were examined to identify specific sites (e.g., intersections) within the mercator, and these were retained as the sites for study. Victims from accidents occurring at the sites were identified and severity scores calculated. An intersection generally included 200 on each approach as attributed to the intersection.

Weighted injury rates for the 100 highest-value weighted frequency mercators were calculated by dividing the weighted injury frequencies by the vehicle-kilometers of travel for the 4 years of exposure. An estimate of 1.0 km as the length of numbered highway in each mercator was used for this calculation. Traffic volume estimates were those for 1986 applied to each of the 4 years: these were supplied by the Ministry of Transport of Quebec. Weighted rates are expressed as weighted frequencies per 100 million vehicle-km of travel. Figure 1 presents the results of the first stage in this selection process. Weighted rates and frequencies are plotted for the 100 mercators with the highest-weighted frequencies. This model of presentation is based on the work by Barbaresso in 1981 (*16*).

The consideration of injury rates and injury frequencies represents different and generally opposing perspectives for the identification of dangerous sites. Rates reflect a measure of risk for the individual roadway user for a given road; frequencies reflect the overall accumulated societal (collective) cost of injuries for a particular site. Rational investment of limited resources for maximal societal benefit will prioritize the examination of sites with high injury frequencies, all other elements being equal; however, considerations of equity and risk reduction for individual users require attention to limit disparities in rates. As seen in Figure 1, even though only the 100 highest-frequency mercators are included in the figure, high-frequency mercators are usually those with greater traffic volumes (i.e., Interstates and Routes 100 to 199) whereas high rate mercators are those with less traffic (Routes 200 to 400).

Using explicit criteria for both rates and frequencies, we identified 56 mercators in three priority groups for further study (Table 5). A fourth group of 44 mercators with weighted frequencies of less than 250 and weighted rates of less than 1,000 per 100 million vehicle-km were eliminated from further study.

The second stage of identification of specific sites within mercators was done by examining printouts of locations for injury accidents within each of the 56 mercators retained for study.

It was possible to identify specific intersections for many of these mercators, but in other cases this was not readily apparent. For those, the entire mercator was retained and identified as a dangerous section at this stage of analysis.

**TABLE 4 Concordance of 50 Highest-Frequency Mercators Determined Using Four Injury Severity Indexes and Before- and After-Correction Accident Location Data**

| After correction \ Before correction | Severity index | | | |
|---|---|---|---|---|
| | Index 1 (10-9-3) | Index 2 (10-9-0) | Index 3 (100-20-3) | Index 4 (100-200) |
| . Severity index 1 | 41 | 32 | 36 | 32 |
| . Severity index 2 | 29 | 28 | 27 | 27 |
| . Severity index 3 | 35 | 33 | 36 | 35 |
| . Severity index 4 | 29 | 29 | 32 | 32 |

Numbers shown are the number of concordant pairs for the 50 highest frequency mercators compared 2 at a time.
Numbers in parentheses refer to the index for (fatal - severe injury - minor injury) victims.

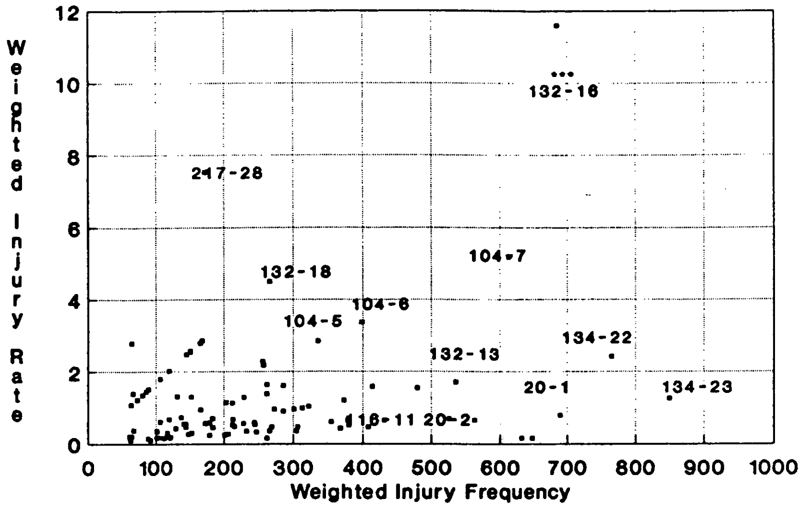



**FIGURE 1 Weighted injury rates and frequencies for the 100 highest-weighted injury frequencies for mercators on numbered highways, territory of DSC Charles LeMoyne, 1984–1987 (*1*).**

**TABLE 5 Number of Mercators Retained for Further Study Identified According to Priority Group**

| Group | Number of mercators codes retained | Definition |
|---|---|---|
| . Priority 1 | 12 | (1) X > 500; or<br>(2) Y > 3 000; or<br>(3) 350 < X < 500 and 2 000 < Y < 3 000 |
| . Priority 2 | 23 | (1) 350 < X < 500 and Y < 2 000; or<br>(2) X < 350 and 2 000 < Y < 3 000; or<br>(3) 250 < X < 350 and 1 000 < Y < 2 000 |
| . Priority 3 | 21 | (1) 250 < X < 350 and Y < 1 000; or<br>(2) X < 250 and 1 000 < Y < 2 000 |

X = weighted injury frequency
Y = weighted injury rate

### Highway Safety-Hazard Inventory

Zegeer et al. reviewed methods for the identification of hazardous highway elements (*3*). Principally on the basis of the model used by the Oakland County (Michigan) Road Commission presented by Zegeer et al., we developed a data collection form for the evaluation of the following roadway elements: fixed objects, guardrails, roadway geometry, signalization, and roadside characteristics other than fixed objects.

A hazard rating for fixed objects and the other characteristics, such as distance from the edge of the road, was defined on the basis of the Oakland study (*15*). Each element was assigned a numeric severity rating based on location and rigidity of the obstacles; ratings ranged from 3.0 to 9.3 and were reduced to three different categories:

- A—most hazardous with scores 7.5 to 9.3,
- B—intermediate level, and
- C—least hazardous, with scores of 3.0 to 4.8.

All 271 km of route were traveled and scored in both directions by two observers, one of whom did the same scoring for two other DSC territories. Identified highway hazards were photographed and a running commentary was tape-recorded to aid completion of the written observation coding sheet. Each hazardous element identified was coded into Categories A, B, or C. A report of hazardous elements for each numbered highway was prepared. Forty-five A-rated sites were identified using this method.

### Community Survey

A community survey was mailed to the 21 local municipalities as well as to community clinics and regional administrations in 1986. Of the 29 respondents, 90 percent thought that the identification and correction of dangerous highway locations was important or very important. Respondents identified 83 sites that they considered dangerous or potentially dangerous; 38 of these sites were on numbered highways.

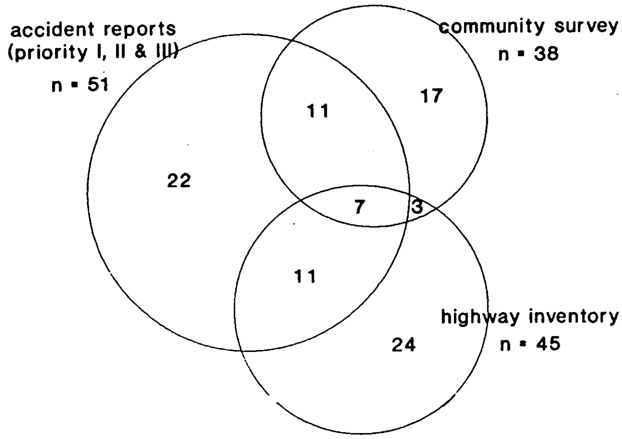
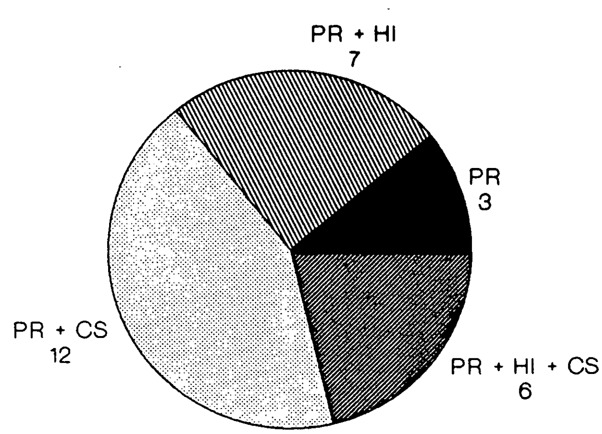



**FIGURE 2 Method of identification of 95 potentially dangerous highway locations according to information source: Priority 1, 2, and 3 mercators identified from accident reports, the community survey, or the highway safety-hazard inventory.**




**PR = Police reports (priority I, II or III).**
**HI = Highway inventory.**
**CS = Community survey.**

**FIGURE 3 Distribution of 28 sites by method of identification, territory of DSC Charles LeMoyne.**

**TABLE 6 Criteria for Selecting Priority Sites**

| Criteria | Number of sites |
|---|---|
| All sites identified within priority 1 **mercators** (table 5) | 12 |
| All sites identified within priority 2 **mercators** (table 5) for which one or the other of the following criteria apply: . they have a highway inventory hazard code of A, or; . they were identified in the community survey | 14 |
| All sites identified within priority 3 **mercators** (table 5) for which both of the following criteria apply: . they have a highway inventory hazard code of A, and; . they were identified in the community survey | 2 |

### Integration Method for Selecting Priority Sites

Overall, the three methods identified 95 different sites on numbered highways. Figure 2 presents these sites according to the method by which they were identified. Most of the sites were identified by only one method; 32 sites (34 percent) were identified by two or three methods.

The criteria in Table 6 were applied to select the final 28 sites retained as priority sites. It should be stressed that all of the final sites identified as high priority were selected from the 56 mercators defined in Table 5.

## RESULTS

The evaluation retained 28 sites as high priority for further study. Three sites were on interstates, 22 were on principal highways (numbered highways 100 to 199), and 3 were on secondary highways (numbered highways 200 to 399). The total combined length of the 28 sites is 17.6 km, or 6.3 percent of the 271 km of numbered highways studied. Fifty-four deaths, 169 severe injuries, and 1,084 minor injuries were attributed to these sites; these represent 53 percent of total deaths, 30 percent of severe injuries, and 32 percent of minor injuries attributed to the 271 km of numbered highways studied.

The methods of identification of the 28 sites are presented in Figure 3. Six sites were identified by all three methods. Seventeen sites are in rural locations, and 11 are in urban areas. All three interstate sites are at interchanges, and five of the sites on other numbered highways are at intersections. Eighteen other sites are defined as highway sections less than or equal to 1 km long and may include several intersections.

The weighted rates and frequencies of injuries for each of the 12 numbered highways included in the study are presented in Figure 4.

The hazardous features and injury experience attributed to each of these sections is presented in our final report. Frequently identified hazards include poorly maintained and poorly aligned guardrails, usually not in continuity with bridge abutments; poorly maintained highway shoulders; and deficiencies in highway geometry for some highways (particularly Route 104). Additional features are presented in our regional report (*17*).

## DISCUSSION OF RESULTS

This study presents several features that we think deserve further attention. We are disturbed by the 20 percent increase in the number of fatal and severe-injury accident reports attributed to numbered highways after localization information was corrected. Small numbers of reports are involved, and one numbered route in an urban area contributed an important fraction of the total; nonetheless, in future use of police reports, particularly in areas in which numbered highways pass through larger urban areas, the underidentification of the importance of injury accidents occurring on numbered highways may represent a significant data treatment issue.

The deficiencies of the mercator system used in Quebec to localize accidents have been confirmed in this study; the number of fatal and severe injury accidents with mercators iden-

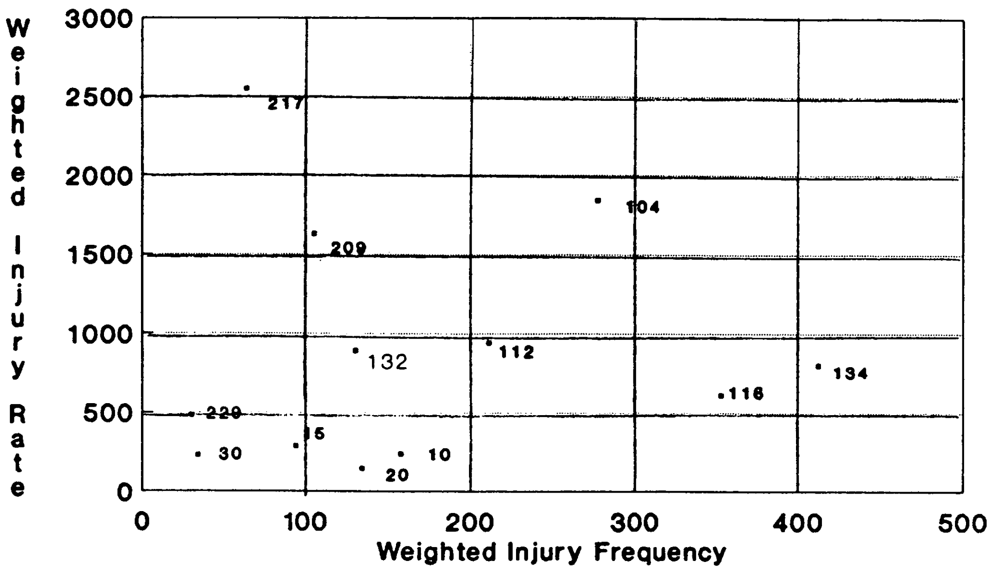



**FIGURE 4 Weighted injury severity rates and frequencies for numbered highways, territory of DSC Charles LeMoyne (*Source: Quebec Automobile Insurance Society and Ministry of Transport of Quebec*).**

tified on numbered highways was increased by 88 percent after our correction procedure. It should, however, be noted that our correction procedure was simple and inexpensive. Computer printouts of written location information are available for all reports, and we basically corrected the 1,300 reports in 2 days, after which printouts were sent to participating municipal and provincial police in our area. The process of working with police was positive. Thus, despite data inadequacies, corrections at a local level are generally quite straightforward at this level of precision.

We believe that our use of injury victims as the unit of analysis is innovative and has some advantages in accident severity counting, when viewing traffic safety from a public health perspective in the context of a screening study. These were discussed earlier. One disadvantage is the possibility of identifying false positives; for example, a single fatal accident involving six deaths would get undue attention.

The use of both frequencies and rates of injury is, we believe, a positive aspect of this study. The decision-making process used to establish priority groups was largely intuitive, however, and a more statistically sophisticated stratification decision analysis would be useful.

Our highway inventory methodology is quite straightforward and feasible for local highway analysis. We perceive the level of precision of our measurements to be low, although appropriate as a screening tool. We have had no evaluation of interobserver reliability nor of the validity of our measurements.

Our decision to limit the number of sites for further study to 28 was defined by our perception that a larger number would overload the capacity of local agencies to study the sites. This corresponds to the recommendation of the panel reviewing highway accident analysis systems; according to this report 1 man-year was required to analyze and review 170 sites for the California Department of Transportation in 1978 (*2*).

Overall, we think that the approach we have chosen is a useful pilot project that contributes to our ability to identify dangerous highway locations systematically. Work on this project was done over 4 years and involved four health departments without external funding. The total cost for the development and application of the method for the entire region (Monteregie) was less than $150,000 (Canadian), or about $9,000/year per health department; about one-third of costs were for development of the methodology, including the initial experimentation in one subregion of the Monteregie (*17*). The total contribution including development costs for the territory covered in this paper (DSC Charles LeMoyne) was about $32,000 (Canadian).

Since spring 1990 we have been working with the Ministry of Transport of Quebec, the Ministry of Municipal Affairs, local municipalities, municipal and provincial police, and elected municipal councillors in a pilot project studying all accident reports, including all material-damage reports, for the period 1986–1990 as well as remediable factors related to vehicles, human factors, and the roadway for eight sites across the Monteregie. The model used is based loosely on the Local Highway Improvement Program of FHWA with additional attention to human and vehicular factors contributing to injury frequency and severity (*18*).

## ACKNOWLEDGMENTS

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# Quantitative Examination of Traffic Conflicts

HOONG C. CHIN, SER T. QUEK, AND R. L. CHEU

Traffic conflict studies have been undertaken in many countries to examine the level of safety on road intersections. Most of these studies involve some form of conflict counts based on rather subjective observations of traffic interactions. An objective way of defining conflicts is proposed along with two conflict measures. Instead of relying on conflict counts, the method uses the probability distribution of the conflict severity measures to derive the probability of a serious conflict. To do this, the severity of each conflict is obtained by first identifying the most serious instant of conflict occurrence according to the proposed measures. The method was applied by examining the relevant data from traffic movements at a merging area on an expressway.

For many years, accident statistics have been used to assess the safety level of roads and to evaluate road safety programs. The lack of good and reliable accident records in some cases has hampered proper analyses. To overcome this problem, attempts have been made to rely on nonaccident statistics. In a landmark paper published in 1968, Perkins and Harris of General Motors Corporation introduced the concept of traffic conflicts as a surrogate measure of accidents (*1*). Since then, studies have been undertaken in several countries to apply the traffic conflict techniques in analyzing the accident potentials at specific road intersections and interchanges (*2–5*).

One of the main problems encountered in most conflict studies is in defining the conflicts and hence developing the procedure for detecting conflicts. Perkins and Harris considered conflicts to be cases of vehicle interactions in which one of the vehicles takes evasive actions, such as braking or swerving (*1*). Such a definition requires, to a large extent, the subjective judgment of the observers. This is clearly unsatisfactory and has led to a wide range of measures of expressing conflicts and varied methods of making conflict observations. A general definition of conflict was finally agreed on at the First International Traffic Conflict Techniques workshop (*2*): a traffic conflict was considered to be "an observable situation in which one or more road users approaches each other in space and time to such an extent that a collision is imminent if their movements remain unchanged."

Even with this definition, the procedures of conducting traffic conflict studies adopted by various countries, along with the criteria for identifying and classifying conflicts and the methods of making conflict observations, remain varied. Most of the studies still rely very much on subjective measurement of conflicts, and this has made comparative studies difficult. This problem prompted a major calibration study (*6*) aimed at comparing the different observational techniques in use including a quantitative method of analysis (*7*).

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The search for an objective and quantitative definition of conflict began as early as 1971 when Hayward suggested the use of time-measured-to-collision as an indicator of the risk of a collision (*8*). The time-measured-to-collision, or time-to-collision (TTC), is the time taken for the following vehicle to collide with the leading vehicle if both vehicles continue in the same path without changing their speeds. This measure requires the two vehicles to be on the same path, such as when merging. The presence of conflict is not so obvious when one of the vehicles changes lanes. There are two important modifications to this definition: the minimum TTC ($TTC_{min}$) and the TTC at braking ($TTC_{br}$). The former is the minimum value of TTC obtained in an evasive maneuver, and the latter the value of TTC at the onset of braking of the following vehicle. TTC at the onset of braking is very much similar to the time-to-accident (TA), which is the time taken from the moment one vehicle initiates evasive action to the time of collision if no evasive action is taken (*9*).

In cases in which vehicles are crossing each other's paths, TTC may be infinite even when the collision is just avoided. Allen has proposed the use of postencroachment time (PET) to measure conflicts (*10*). This is the time difference between the arrival of the conflicted vehicle and the departure of the offending vehicle at the point of crossing. Although PET can be objectively measured, it is uncertain whether its magnitude truly represents the severity of the conflict or the willingness of the drivers in accepting the risk. This is because the most serious conflicts may have rather large PET values if evasive actions have been taken.

One way of overcoming this is to consider the gap time (GT), which is the difference between arrival times of the involved vehicles at the point of crossing if no evasive actions are taken by either vehicle (*11*). Glauz and Migletz have argued that this may indicate whether a potential conflict exists, but it is by no means a perfect measure of the severity of conflict since a zero-value GT can be recorded in two possible cases: one that involves an accident and the other in which early precautionary actions are taken (*12*).

Another objective measure for vehicles approaching an intersection is the time-to-intersection ($TTI_{br}$), which is the time expected for a vehicle to enter the intersection at the constant instantaneous speed just at the onset of braking (*13*). This has been used for single-vehicle interaction at nonsignalized intersections (*10*).

Given that conflicts can be measured objectively and quantitatively, it is still necessary to determine a threshold value to distinguish a conflict serious enough to be detected. It is relatively simple to visualize and define the case of collision since all the quantitative measures must take on definite values (zero for TTC, PET, TTI, and GT). On the other hand, it is not so simple to specify a threshold value for a serious conflict or a near-collision. Various threshold values have been assumed for the different measures of conflict. It has been assumed that TTC can be related to the drivers' reaction times. Consequently, values of 0.5 to 1.5 sec of TTC have been used to define instances of near-collision (*8,14–16*). Some have assumed the threshold values to vary with the speeds of the vehicles (*15*). A value of 1.5 sec has been adopted by Hydén for TA (*17*), but he later assumed the threshold values to vary with speeds (*18*). For PET, values from 0 to 4 sec have been used to define the different levels of conflict severity (*10,11,19*). Values between 1.5 and 3.0 sec have been used for TTI in situations of vehicles yielding at nonsignalized intersections (*13*).

The foregoing indicates that attempts have been made to express conflicts quantitatively. However, many conflict studies still end up observing conflict counts on the basis of rather imprecise definitions of conflicts (*2–5*). In this paper, an objective way of defining conflicts is proposed along with two conflict measures, one related to TTC and the other to deceleration. Instead of making conflict counts, the method uses the probability distribution of the conflict measures to derive the probability of a serious conflict. Furthermore, since conflict encounters are really processes instead of events, the severity of each conflict is obtained by examining the proposed conflict measures continuously. To apply this technique, traffic movements at a merging area on an expressway were filmed using video cameras. The relevant data were then extracted by playing back the films in the laboratory.

## STUDY METHOD

### Derivation of Conflict Severity

Consider a situation on an expressway in which a pair of vehicles are involved in a merging process (one is merging and one is on the expressway). A possible conflict exists when the offending (merging) vehicle shares the same path as the conflicted (mainline) vehicle over a certain period of time. Suppose that at time $t$, the merging vehicle and the mainline vehicle are respectively at positions $x_m(t)$ and $x_e(t)$ downstream from the ramp nose on the expressway and at speeds $v_m(x)$ and $v_e(x)$ where $x_e(t) < x_m(t)$ (see Figure 1). We may also denote the time at which the merging vehicle to be at a specific point $x$ on the expressway as $t_m(x)$ and the time for the mainline vehicle to be $t_e(x)$. Taking the physical length of the vehicles into consideration, we may consider $t_m(x)$ to be measured with reference to the rear bumper of the vehicle and $t_e(x)$ with reference to the front bumper of the vehicle.

In this study, two conflict measures are proposed; one related to the TTC and the other to the deceleration of the conflicted vehicle. TTC depicts the time proximity between vehicles before collision if both vehicles continue along the same path with unchanged speeds. However, as the severity of conflicts increases with decreasing values of TTC, it seems more appropriate to define a conflict measure as the reciprocal of TTC, that is,

$$c_1 = \left[\frac{v_e(x) - v_m(x + \Delta x)}{\Delta x(x)}\right] \tag{1}$$

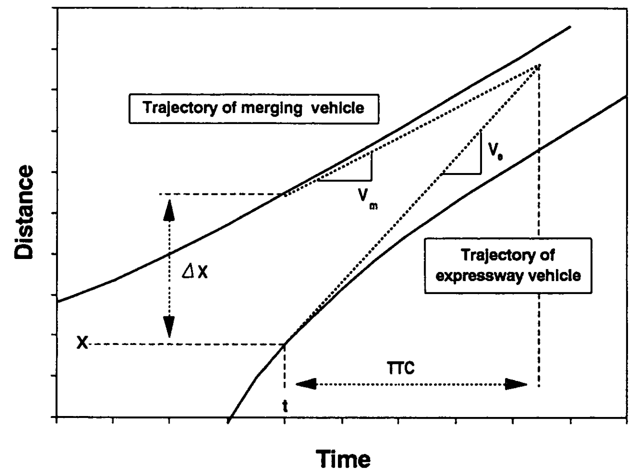


**FIGURE 1 Space-time trajectories of vehicles.**

where

$$\Delta x(x) = x_m[t_e(x)] - x \tag{2}$$

For a particular merging encounter, the value of $c_1$ computed for that pair of vehicles will change continuously during the entire process of merging. It is obvious that the most serious instant of conflict between the pair of vehicles occurs when $c_1$ is at a maximum or when TTC is at a minimum. Consequently, the severity of conflict $s_1$ for that merging as measured by $c_1$ will be

$$s_1 = \max_x \{c_1\} \tag{3}$$

The second conflict measure is associated with the magnitude of the average deceleration that the conflicted vehicle is required to take just to avoid a collision. Provided that TTC is positive, the mainline vehicle will avoid a collision if its speed can be reduced to that of the leader by the application of a constant deceleration. The second proposed measure defined as the deceleration to avoid a collision is given by

$$c_2 = \left\{\frac{|v_e(x) - v_m(x + \Delta x)|\,[v_e(x) - v_m(x + \Delta x)]}{2\Delta x(x)}\right\} \tag{4}$$

As in the previous measure, the corresponding severity of the conflict defined by the second measure will be

$$s_2 = \max_x \{c_2\} \tag{5}$$

### Evaluation of Conflict Probability

The conflict measures as defined in Equations 1 and 4 imply that a conflict exists only when $c > 0$. Equations 3 and 5 also signify that the maximum instantaneous conflict value in any merging process represents the severity of conflict of the merging encounter. Suppose an appropriate threshold value for the severity of the conflict, $s^*$, can be identified. Then it is also possible to clearly distinguish the serious conflicts objectively by considering quantitatively the cases in which $s > s^*$. Traditionally, in most conflict studies, one would determine the probability of occurrence of a serious conflict by simply noting the proportion of cases in which $s$ exceeds $s^*$.

Another method is used here. Because the conflict measures are well defined, it seems more appropriate to use as much information as possible from the data gathered instead of limit the analysis to obtaining counts of serious conflicts. Logically, the two proposed measures of conflict severity should follow some probability distribution, and it is possible to obtain a suitable mathematical distribution to describe $s$, from the data gathered for the two measures. Hence, if the positive values of $s$ follow a probability density function $g(s)$, then the cumulative distribution function of $s$, $F(s)$, may be defined as

$$F(s) = p_0 + (1 - p_0)\int_0^s g(z)dz \qquad s > 0 \tag{6}$$

where $p_0$ is the probability that $s$ is negative. The probability of a serious conflict may be derived given the threshold value $s^*$. However, the ability to avoid a collision is very much dependent on the drivers and their vehicles, which means that the value of $s^*$ is not likely to be unique in general. Supposing that the threshold follows a probability density function $h(s^*)$, then the probability of the occurrence of a critical conflict will be

$$p_s = \int_{s=0}^{\infty} [1 - F(s)]h(s)ds \tag{7}$$

When TTC is used to measure conflicts, the threshold selected to distinguish serious conflicts has often been taken to be a function of the driver's reaction time. A single value of the threshold ranging from 0.5 to 1.5 sec has been employed (*13–17*). In this study, the driver's reaction time is also used in conjunction with the first measure of conflict severity. However, instead of relying on a single value of the threshold, a distribution of $s_1^*$, that is, $h(s^*)$, is used; this may be suitably derived from the distribution of driver's reaction time.

For the second conflict measure, a serious conflict is one in which the deceleration needed is excessive for comfort and safety. Since drivers and passengers can comfortably tolerate quite a high level of deceleration, especially if it is over a very short period, it is more appropriate to select a threshold on the basis of safety considerations. At high deceleration, the driver loses control of the vehicle if the braking force exceeds the skidding resistance between the tires and the pavement. Hence, if the critical conflict is considered to be one in which the vehicle will skid on the road surface should the driver brake excessively to avoid a collision, an appropriate distribution of the threshold would be the distribution of the skid resistance between the tires and the road surface.

## EXPERIMENTAL DATA

To examine the suitability of the proposed conflict measures, traffic maneuvers at the Paya Lebar on-ramp into the westbound direction of the Pan Island Expressway were monitored (*20*). At this merging area (see Figure 2), the acceleration lane—which is aligned at a horizontal angle of 3 degrees to the expressway—is about 100 m long, tapering from a width of 5.8 m at the ramp nose. The nearside lane of the three westbound lanes on the expressway is 3.8 m wide. The expressway at this point has a straight horizontal alignment and a 3 percent downgrade about 100 m upstream of the ramp nose. Traffic interruptions due to geometric changes are unlikely, if at all possible, because the geometric features of the sections immediately upstream and downstream of the merging area are generally consistent with those of the merging area.

Most of the vehicles merging into the expressway at this location do so within the first 50 m from the ramp-nose, so it is sufficient to observe traffic maneuvers within the 100-m stretch downstream of the expressway and on-ramp with reference to the ramp nose. The movements of the vehicles within this study area were recorded with video cameras from a tall building nearby for recording periods of about an hour. Taking into account the variation in traffic volumes during the day, eight recording periods were made so that both peak and off-peak conditions during daylight were covered. The time periods during which the data were obtained are shown in Table 1.

In order to obtain the space-time relationships of each pair of vehicles involved in the merging encounter, markers at 10-m intervals were set up on both sides of the expressway. From these markers, 11 lines across the expressway and the ramp were constructed on a 100-in. screen in the video playback. The arrival times of the vehicles $t_m(x_i)$ and $t_e(x_i)$ at Marker $i$ were then extracted from the video playback, which

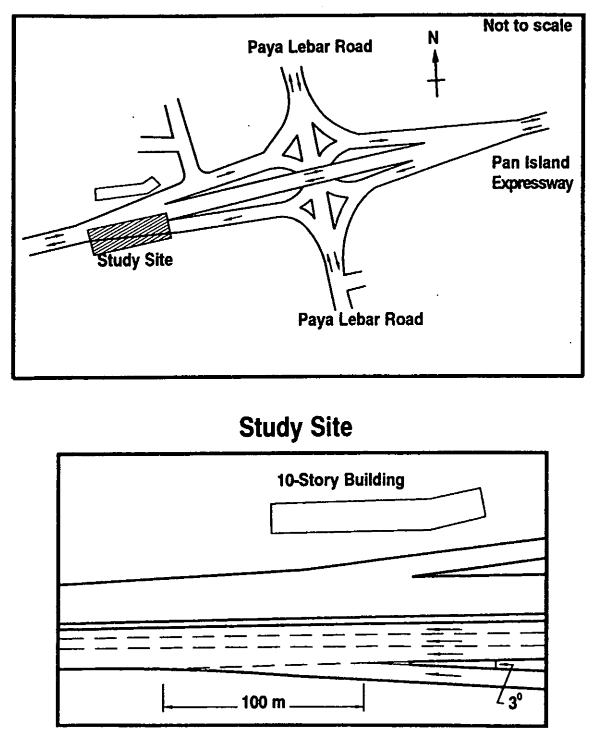


**FIGURE 2 Location of study site (not to scale).**

**TABLE 1 Information on Sets of Data Used in Analysis**

| Data set | Date | Time |
|---|---|---|
| Set 1 | 8 Jun 88 | 0700-0800 |
| Set 2 | 8 Jun 88 | 0800-0900 |
| Set 3 | 15 Apr 88 | 0900-1100 |
| Set 4 | 5 May 88 | 1130-1230 |
| Set 5 | 5 May 88 | 1230-1330 |
| Set 6 | 4 May 88 | 1500-1700 |
| Set 7 | 28 Sep 88 | 1700-1800 |
| Set 8 | 28 Sep 88 | 1800-1900 |

was run on a slow speed of 2.5 frames per second to achieve the desired accuracy in the arrival times. A controlled study was also undertaken to minimize the errors of measurement and observer bias (*20*).

## ANALYSIS OF CONFLICT DATA

On the basis of the space-time data extracted from the video films, the kinematics of the vehicles involved in the merging process can be derived. These data form a useful data base for investigating the mechanics of vehicle interaction during merging. In particular, it is possible to determine for each merging encounter the values of the proposed conflict measures $c_1$ and $c_2$ as given in Equations 1 and 4.

### Reciprocal of TTC as First Conflict Measure

The use of TTC in describing a conflict implies that a conflict exists only when the expressway vehicle is traveling at a higher speed than the merging vehicle. To observe how the reciprocal of TTC—that is, $c_1$—varies in a merging process, a few merging encounters are presented as typical examples. As seen in Figure 3, when the interaction between vehicles results in little or no danger of collision, the variation of $c_1$ is small and fluctuates around the zero level. As the severity of the conflict increases, so does the fluctuation. Where a precautionary action is taken so that a serious conflict is avoided, a slight dip in $c_1$ is observed. For a serious but short conflict in which definite corrective actions are taken, the drop in $c_1$ can be considerable.

By considering the maximum of $c_1$, that is, $s_1$ in each case of merging for a particular time period observed, it is possible to establish the distribution of $s_1$. For each period of observation, various mathematical functions have been tested to fit $g(s)$ in Equation 6. The function for $g(s)$ that most suitably fits the empirical data is found to be the Weibull distribution, a largest-value extremal function given by

$$g(s) = \left[\frac{k}{w}\right](s/w)^{k-1}\exp[-(s/w)^k] \qquad (8)$$

The parameters of the Weibull distribution and the goodness-of-fit statistic as judged by the Kolmogorov-Smirnov test are presented in Table 2 for each of the periods studied. The results indicate that the data fit the Weibull distribution well. A typical distribution of $s_1$ along with the best-fit Weibull distribution is plotted in Figure 4.

**TABLE 2 Parameters of Weibull Distribution, Goodness-of-Fit Value, and Computed Conflict Probabilities for $s_1$**

| Data set | $p_o$ [a] | k [b] | w [b] | $D_c$ [c] | $p_{s1}$ ($\times 10^{-3}$) [d] |
|---|---|---|---|---|---|
| Set 1 | 0.180 | 1.080 | 0.153 | 0.065 | 0.538 |
| Set 2 | 0.184 | 1.238 | 0.184 | 0.054 | 0.512 |
| Set 3 | 0.137 | 1.289 | 0.156 | 0.037 | 0.126 |
| Set 4 | 0.256 | 1.007 | 0.115 | 0.024 | 0.185 |
| Set 5 | 0.319 | 1.079 | 0.104 | 0.036 | 0.040 |
| Set 6 | 0.276 | 1.054 | 0.146 | 0.064 | 0.450 |
| Set 7 | 0.245 | 1.154 | 0.135 | 0.045 | 0.124 |
| Set 8 | 0.324 | 1.064 | 0.118 | 0.030 | 0.111 |

[a] Proportion of non-conflicts
[b] Parameters of Weibull distribution (Eq. 8)
[c] Kolmogorov-Smirnov test value
[d] Probability of serious conflict

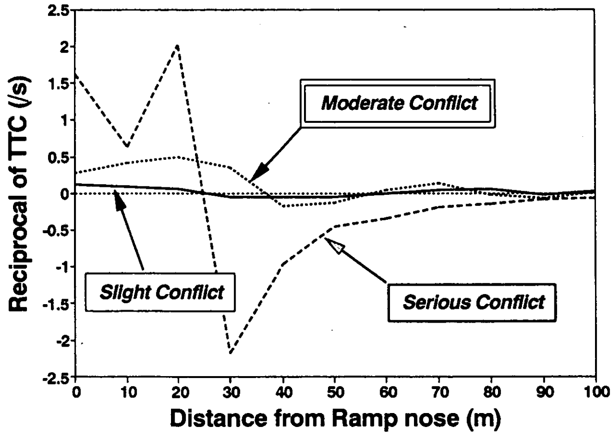


**FIGURE 3 Variation in conflict measure $c_1$ during typical merging encounters.**

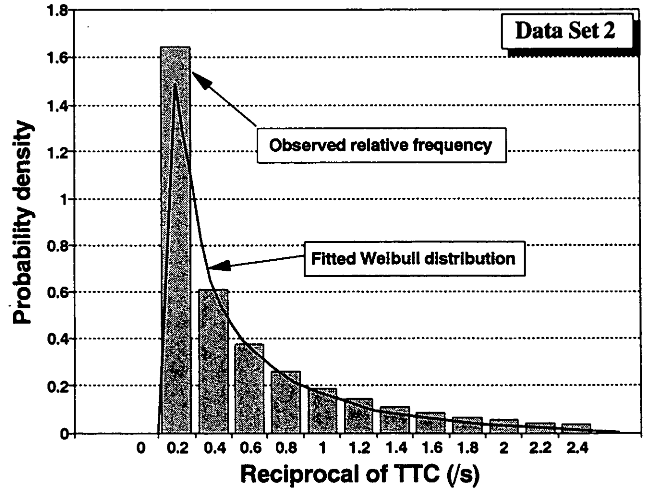


**FIGURE 4 Distribution of severity measure $s_1$.**

To obtain the probability of a critical conflict, the distribution of drivers' reaction times reported by Johansson and Rumar (*21*) as reproduced in Figure 5 is used to derive $h(s_1^*)$. Applying numerical integration to Equation 7 and using the best-fit Weibull distribution derived earlier for Equation 6, the probability of a critical conflict for each period can be determined and shown in Table 2. The values of $p_{s1}$ computed for the different periods vary from 0.000040 to 0.000538. Because the probability estimates are small, it may not be appropriate to compare the values of $p_{s1}$ numerically. It may be best just to consider that $p_{s1}$ is of the order of magnitude of $10^{-4}$.

**Deceleration To Avoid Collision as Second Conflict Measure**

Comparison between $c_1$ and $c_2$ in Equations 1 and 4 shows that $c_2$ is a weighted function of $c_1$. Therefore, the variation of $c_2$ during a merging process will be quite similar to that of $c_1$ as seen in Figure 6 in relation to Figure 3 for the same sets of vehicles. Again, in the more serious cases of conflict, the variation in $c_2$ is high. Compared to $c_1$, the variation of $c_2$ is more pronounced at higher values of $c_2$. The effect of this is a greater spread in the distribution of $s_2$.

The data values of $s_2$ have also been used to fit to a number of mathematical distributions, and the Weibull distribution again gives the best fit. The parameters of the distribution are shown in Table 3 with the Kolmogorov-Smirnov goodness-of-fit statistics. Although the Weibull distribution is acceptable in describing the distribution of $s_2$ for all the data sets, compared with $s_2$ it appears not to fit as well. Figure 7 shows the distribution of $s_2$ and the fitted Weibull distribution for the same set of observations that are used to generate the distribution of $s_1$ in Figure 4.

To obtain the distribution of skid resistance, the British pendulum tester was used at the site under dry pavement conditions as a measurement of the coefficient of static friction between the tires and the pavement. The mean British pendulum number obtained from 12 points in the study site was 65.2, and the standard deviation was 6.2. Using the exponential model of skid variation proposed by Shah and Henry

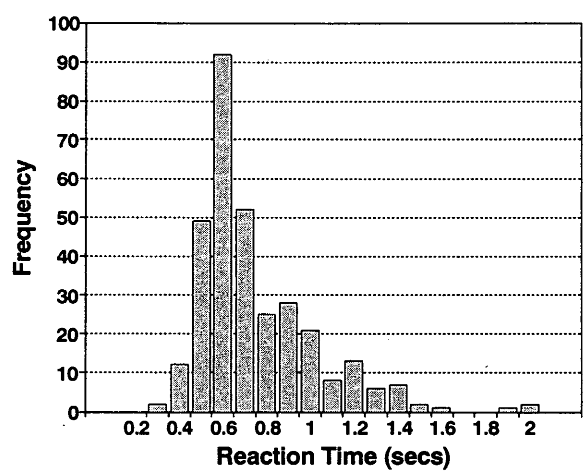


**FIGURE 5 Distribution of drivers' reaction times (*21*).**

**TABLE 3 Parameters of Weibull Distribution, Goodness-of-Fit Value, and Computed Conflict Probabilities for $s_2$**

| Data set | $p_o$ [a] | k [b] | w [b] | $D_c$ [c] | $p_{s2}$ (X$10^{-3}$) [d] |
|---|---|---|---|---|---|
| Set 1 | 0.180 | 0.585 | 0.167 | 0.077 | 0.549 |
| Set 2 | 0.184 | 0.707 | 0.304 | 0.058 | 0.606 |
| Set 3 | 0.137 | 0.675 | 0.231 | 0.047 | 0.310 |
| Set 4 | 0.256 | 0.572 | 0.161 | 0.038 | 0.601 |
| Set 5 | 0.319 | 0.592 | 0.115 | 0.034 | 0.065 |
| Set 6 | 0.276 | 0.582 | 0.158 | 0.067 | 0.419 |
| Set 7 | 0.245 | 0.632 | 0.165 | 0.051 | 0.138 |
| Set 8 | 0.324 | 0.628 | 0.163 | 0.037 | 0.131 |

[a] Proportion of non-conflicts
[b] Parameters of Weibull distribution (Eq. 8)
[c] Kolmogorov-Smirnov test value
[d] Probability of serious conflict

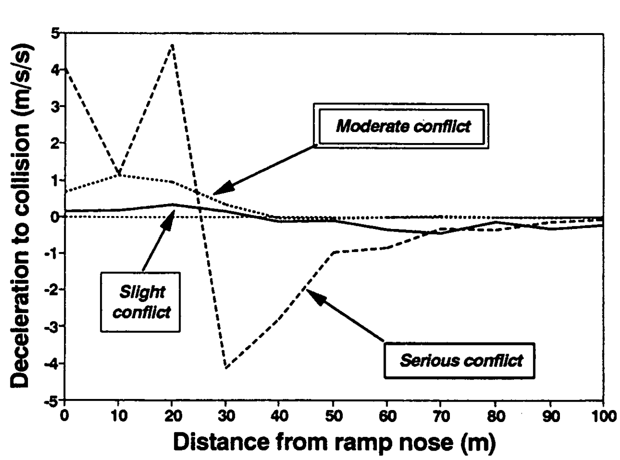


**FIGURE 6 Variation in conflict measure $c_2$ during typical merging encounters.**

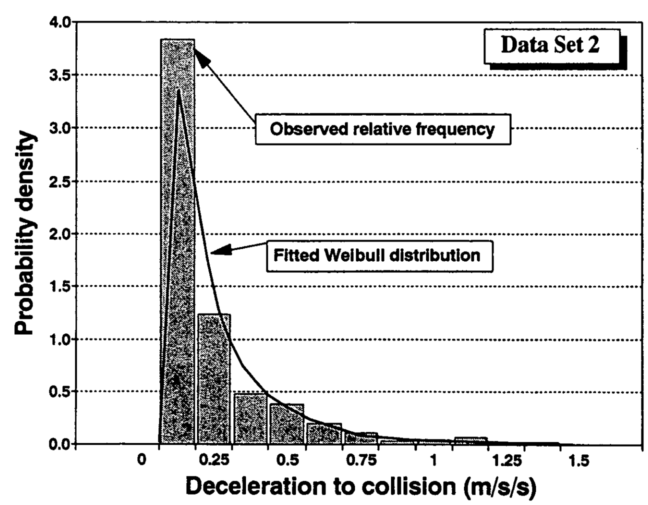


**FIGURE 7 Distribution of severity measure $s_2$.**

(22) and a measured average speed of expressway vehicles of 49.3 km/hr, the coefficient of braking friction works out to a mean of 0.52 and a standard deviation of 0.05. Taking the coefficient of braking friction to be normally distributed and the threshold $s_2^*$ to be the braking skid resistance, it is possible to derive $h(s_2^*)$. By applying the best-fit Weibull distribution to Equation 6 and the distribution with $h(s_2^*)$ to Equation 7, the probability of a critical conflict, $p_{s2}$, can then be determined for all the periods. From the computed values of $p_{s2}$ shown in Table 3, it can be seen that $p_{s2}$ ranges from 0.000065 to 0.000606, giving slightly higher values than $p_{s1}$. As in the first conflict measure, $p_{s2}$ may be considered to be of the order of $10^{-4}$. A comparison between $p_{s1}$ and $p_{s2}$ shows that in general $p_{s2}$ is about 1½ times larger than $p_{s1}$. It may be argued that a larger $p_{s2}$ is not surprising because the chances for skidding are likely to be higher than those for collision.

## CONCLUSIONS

Using two ways of defining conflicts, this paper illustrates how the probability of a serious conflict can be determined. This method differs in several respects from a number of other conflict studies in the manner by which conflicts are studied. First in this study, conflicts are examined objectively using quantitatively measurable observations. Second, the severity of a conflict is not measured at a particular point in space or time but rather determined by examining the process of vehicle interaction and identifying the most serious instant of conflict. Third, rather than relying on mere conflict counts, which requires only a simple "yes" or "no" treatment of conflict observations, the proposed method uses the full range of observations to determine the distribution of conflict severity. Finally, the threshold to identify the critical cases of conflict is taken to be a distribution instead of a single value.

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