855

TRANSPORTATION RESEARCH RECORD 855

# Visibility and Operational Effects of Geometrics



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# Visibility and Operational Effects of Geometrics

TRANSPORTATION RESEARCH BOARD

NATIONAL RESEARCH COUNCIL

NATIONAL ACADEMY OF SCIENCES
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# Legibility of Highway Guide Signs

DONALD A. GORDON

**A study concerned with the legibility of the message elements displayed on highway guide signs is reported. The work was carried out on a vision testing alley, and scaled-down replicas of highway signs were used. The findings indicate that route numbers had the poorest legibility of the eight types of information displayed on the guide signs tested. They were seen at 10 percent shorter distance than place names. Cardinal-direction indications (North, South, East, West) demonstrated satisfactory legibility. They could be identified farther away than any of the sign elements except the message "NEXT RIGHT". Capital/lower-case cardinal-direction indications were seen 10 percent farther away than conventional block letters. The use of capital/lower-case lettering can increase element legibility without requiring appreciably more sign space. The feasibility of performing sign legibility research on small-scale replicas is supported. A minimum letter height of 0.85 cm (0.33 in) is recommended to achieve adequate subject testing distance.**

The study reported in this paper was initiated by a request to the Traffic Systems Division from the Office of Traffic Operations of the Federal Highway Administration (FHWA). Motorists had complained of difficulties in reading the cardinal directions (North, East, South, West) on highway signs. On the basis of subsequent discussions, it was decided that a study of cardinal-direction legibility should be undertaken. It was thought that legibility might be enhanced by capitalizing and increasing the size of the first letters of the directions. South and North have the letters o, t, and h in common; East and West both include the letters s and t. However, the first letters--N, E, S, and W--are unique to each direction. By increasing the size of the first letters, the uniqueness of the directions could be enhanced. The legibility of guide-sign categories other than cardinal directions was also tested in this study. These elements included such categories as place names, route numbers, exit numbers, and the warning, "EXIT ONLY". Since complete sign replicas were made to test the legibility of cardinal indications, with a little additional effort the legibility of other sign elements could also be assessed.

This study differs from previous work (1) in that it is concerned with the legibility of the elements of a complete sign rather than single isolated words or letters. Recognition of the sign elements may be affected by cues of familiarity, position, color coding, and word sequence. For example, if the driver identifies either word of the sequence "EXIT ONLY", he or she is likely to guess the entire element. Position on the sign and color coding also aid this identification. If the driver knows that he or she is viewing a direction indication, the choice is only among the four cardinal directions. Because these effects are present in the operational situation, it is of interest to study legibility in context, by using the complete sign.

## METHODOLOGICAL CONSIDERATIONS OF SIGN TESTING ON SMALL-SIZED REPLICAS

In this study, legibility was tested on small-sized replicas rather than on full-scale signs. The advantages of using replicas are readily apparent. A 38.4-cm (15-in) high word, such as appears on highway guide signs, can be read at a distance of 305 m (1000 ft) or more by a person with normal vision. A legibility study carried out with full-scale letters would require, in addition to this distance, a preliminary range to prevent instant word recognition by subjects with especially keen eyesight. Full-sized signs also require cumbersome display material and a method of communication between the experimenters overseeing the subjects and those manipulating the sign display.

The testing of legibility on small-sized replicas assumes that a subject's acuity, over a large range, can be described by an angular threshold measurement. Legibility distance would consequently be directly proportional to the linear height of the sign lettering. A replica 1/45th the size of the actual road sign, for example, would be readable at 1/45th the legibility distance of the original sign. If these assumptions were not true, and acuity varied with distance, the "walk-to" method of determining legibility would be incorrect. Indeed, optometric acuity testing, which is usually done at a fixed 6-m (20-ft) distance, would have but limited applicability.

Considerable evidence exists that the legibilities obtained on small replicas can be extrapolated to full-sized signs. Visual accommodation is not a problem; ophthalmological studies show that, at distances as close as 1 m (3.28 ft), even the far-sighted person can fully accommodate (2). A study by Forbes and others (3) has demonstrated constancy of threshold visual angle. In the published report of that research, the legibility distances associated with letters 12.7, 20.32, and 30.48 cm (5, 8, and 12 in) in height were separately shown (see Figure 1). The data points in Figure 1 are fitted by lines that depart only slightly from linearity. (A straight-line fit would indicate a linear relation between letter size and legibility distance.) The negligible deviation from straight-line fit is probably accounted for by variable error.

Data on the constancy of the threshold visual angle at short viewing distances have been presented by Smith (4). The Smith study involved 547 subjects who made a total of 2007 legibility observations on 314 types of common printed materials. Results indicated that threshold visual angle did not systematically change between the limits of 1 and 22 m (3.28 and 72 ft). At distances closer than 1 m, the threshold angle did increase, which indicates that the walk-to method may not be reliably applied at these very close viewing positions.

## SUBJECTS

The 50 experimental subjects were recruited from the Virginia Employment Office and from the U.S. Department of Transportation's Fairbank Highway Research Station in McLean, Virginia. The Employment Office subjects were paid $15 for participation in this and an associated study on sign information load. The sample included an almost equal number of males and females and a wide range of ages, education, and driver experience. All subjects held valid driving permits.

## EQUIPMENT

### Testing Alley

The investigation was carried out in a vision testing alley at the Fairbank Highway Research Station. Viewing positions for the Snellen and cardinal-direction charts were marked in tape on the floor at distances of 7.62, 10.67, 12.19, and 15.24 m (25, 35, 40, and 50 ft) from the chart position. The highway sign replicas were tested at the following closer distances (1 m = 3.28 ft):

| Position | Distance from Sign (m) | Position | Distance from Sign (m) |
|---|---|---|---|
| 1 | 1.0583 | 7 | 3.16 |
| 2 | 1.27 | 8 | 3.79 |
| 3 | 1.52 | 9 | 4.55 |
| 4 | 1.83 | 10 | 5.46 |
| 5 | 2.19 | 11 | 6.55 |
| 6 | 2.63 | 12 | 7.86 |

It will be noted that these distances form a geometric series; each step is 1.2 times the distance of the next closer step. A geometric sequence provides subjectively equal size increments in conformity with Webers Law that a perception unit is a constant fraction of the stimulus magnitude. The data, calculated in terms of steps and fractions of a step, may be directly used in the computation of means, standard deviations, and other descriptive statistics.

The signs were displayed against a 56-cm (22-in) high by 71-cm (28-in) wide sheet of white cardboard centered 1.42 m (56 in) above the floor. Illumination of 377 lx (35 ft•c) was furnished by two spotlights at chart height. The lights were sufficiently separated to the front and sides of the charts as not to throw specular reflections into the subject's eyes.

### Test Charts

The tests included a Snellen E chart, cardinal-direction charts, and replicas of highway signs.

## TESTS

The design and administration of the tests were as follows.

### Snellen E Test

A Snellen test was included in order to relate the other test results to the subjects' visual acuities. The Snellen test showed the letter E with the open side facing up, down, right, or left in random sequence. The lines on the chart represented acuities of 20/200, 20/100, 20/70, 20/50, 20/40, 20/30, 20/20, and 20/15. Since subjects viewed the chart binocularly and with corrected vision, scores were better than they would have been had the usual monocular testing been used. To effectively grade keen-sighted subjects who were able to read all lines of the chart, the test was administered at a 9.1-m (30-ft) rather than 6.1-m (20-ft) distance. This procedure allowed the chart to adequately cover the acuities of the tested group.

A 95 percent probability level was adopted in this acuity testing. To pass an acuity level (line) on the test, the subject was required to equal a performance attained only 1 in 20 times by chance alone. The E may appear in any one of four positions; hence, the chance of correctly guessing its position is 1 in 4, or 25 percent. The probability of correctly guessing two consecutive orientations is 1 in 16. The probabilities are therefore given by cumulating the following binomial expansion:

$$(1/4\ A + 3/4\ B)^N$$

where

A = success,
B = failure, and
N = number of letters on a line.

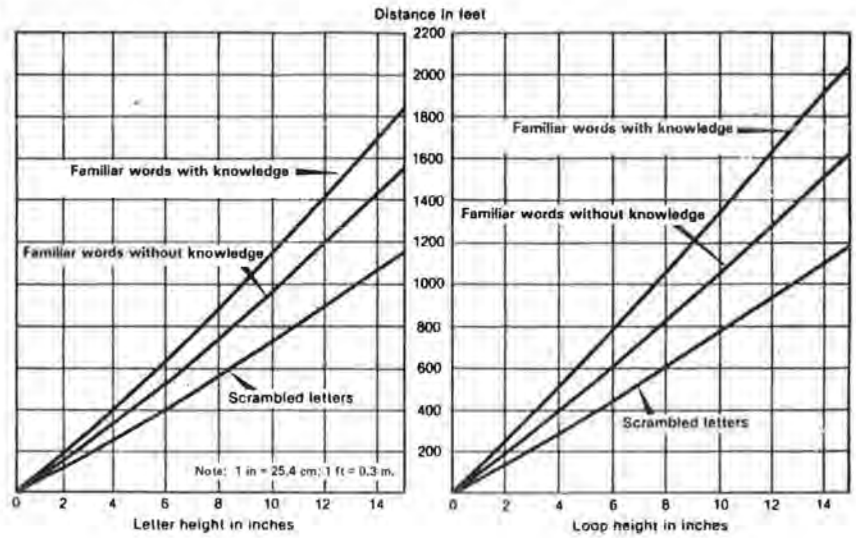
The expansion of the expression indicates that the subject fails at the 0.05 chance level if more than two items are wrong on a six-item line or if more than one item is wrong on a four-item line (there are no five-item lines on the chart).

A person who correctly reads the 20/40 line of an acuity test and fails the next line is able to read at 6.1 m (20 ft) what a normally sighted person can read at 12.2 m (40 ft). Since the test was administered at a 9.1-m (30-ft) distance rather than at 6.1 m, subjects were credited with better vision than the eye chart actually indicated. A subject who read a 20/15 line was graded 20/10. He or she could read at 5.1 m (20 ft) what a normally sighted person would just be able to read at 3.05 m (10 ft). A 20/20 score was classified as 20/13. Similar corrections were made in grading the other line scores.

### Cardinal-Direction Charts

The cardinal-direction charts showed either block or capital/lower-case (CLC) letters (see Figure 2). The directions "North", "South", "East", and "West" were displayed in white letters on a highway green background. The charts were 25.4 cm (10 in) high and 42.5 cm (16.5 in) wide. Each chart had two lines of eight items (two Norths, two Souths, two

Figure 1. Relation between legibility distance and letter height: block and capital/lower-case styles.



Easts, and two Wests) to a line. The conventional block letters were 0.64 cm (0.25 in) in height; the initial capital letters of the other style were 0.87 cm (0.34 in) in height, and the lower-case letters were 0.65 cm (0.255 in) in height. The letter styles duplicated those illustrated in the Manual on Uniform Traffic Control Devices (MUTCD) (5). As shown in the MUTCD, lower-case letters have a somewhat thicker stroke width than do the block letters. Following the rule that a subject passed at a success level exceeded in only 5 percent of trials by chance, the cumulative binomial expansion indicated that a subject failed if more than three mistakes were made on any of the eight item lines.

### Highway Sign Replicas

The four highway sign replicas used in the study were taken from the MUTCD (see Figure 3). The designs were modified slightly: North was never paired with South nor East with West. Subjects were therefore unable to deduce a cardinal direction from its paired opposite on the sign. Two of the signs were conventional exit warning signs, and two were diagrammatic advance warning signs. The block and CLC signs were similar except for the difference in style of the cardinal-direction indications.

## PROCEDURE

The experimental tests were presented in the following order: (a) Snellen E, (b) block and CLC cardinal-direction charts, and (c) highway sign replicas. The 50 experimental subjects were randomly assigned into two groups of 25 subjects each. These groups were designated "experimental" and "conventional", respectively. Both groups took the three types of tests, but the conventional group viewed only the block-letter highway sign replicas. The experimental group viewed the sign replicas with CLC letters. (Since subjects might remember the contents of the sign replicas, only one letter style of replica sign was viewed by each subject.) A counterbalanced order of viewing the cardinal-direction charts was used to offset order effects. It required approximately 15 min for a subject to complete the session.

## RESULTS

### Comparative Legibility of Guide-Sign Elements

It is convenient to describe the sign replica results before the other findings. The median distances at which guide-sign elements were read on each sign are given in Table 1. In computing median

**Figure 2. Cardinal-direction charts.**


**Figure 3. Highway sign replicas used in the study.**

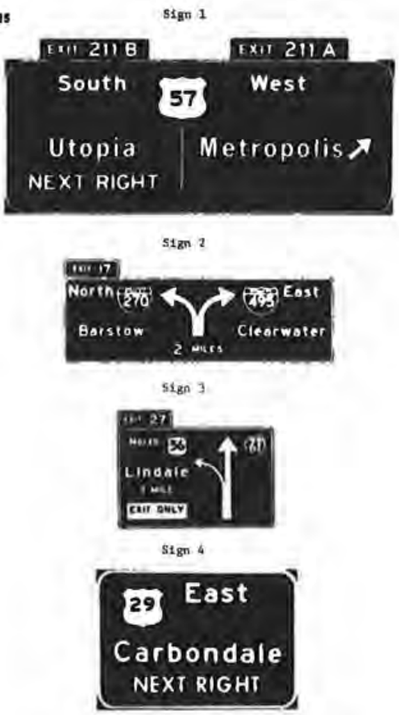



**Table 1. Observed and relative legibility distances of guide-sign elements.**

| Sign Element | Sign 1 Median (m) | Sign 1 Relative Ratio | Sign 2 Median (m) | Sign 2 Relative Ratio | Sign 3 Median (m) | Sign 3 Relative Ratio | Sign 4 Median (m) | Sign 4 Relative Ratio | Geometric Mean | Rank |
|---|---|---|---|---|---|---|---|---|---|---|
| "EXIT" | 3.86, 4.01 | 0.955 | 2.83 | 1.029 | 2.70 | 1.019 | | | 1.001 | 4 |
| Exit number | 3.54, 3.58 | 0.864 | 2.65 | 0.964 | 2.63 | 0.992 | | | 0.938 | 7 |
| Cardinal direction | 4.32, 4.91 | 1.120 | 3.00, 3.27 | 1.140 | 2.48 | 0.936 | 7.02 | 1.376 | 1.132 | 2 |
| Place name | 4.11, 4.13 | 1.000 | 2.67, 2.83 | 1.000 | 2.65 | 1.000 | 5.10 | 1.000 | 1.000 | 5 |
| Route number | 4.00 | 0.971 | 2.60, 2.66 | 0.956 | 2.37, 2.22 | 0.866 | 4.15 | 0.814 | 0.899 | 8 |
| Distance number | | | 2.98 | 1.084 | 2.88 | 1.087 | | | 1.085 | 3 |
| "NEXT RIGHT" | 5.13 | 1.245 | | | | | 5.40 | 1.059 | 1.148 | 1 |
| "EXIT ONLY" | | | | | 2.49 | 0.940 | | | 0.940 | 6 |

Notes: 1 m = 3.28 ft.
Relative legibilities are in reference to median place-name distances, which were given a value of 1.00 (based on legibility of the four guide signs shown in Figure 3, read by 50 subjects).

distances, if a subject passed at a particular position and failed at the next more distant one, he or she was credited with reading the sign element at the geometric mean of the positions. The relative ratio column in Table 1 expresses reading distances of each element divided by place-name legibility distance. Where two similar categories appeared on a sign, legibility distances were averaged. The geometric mean was used in computing the next to last column of Table 1 to ensure that two ratios, such as $4/5 = 0.80$ and $5/4 = 1.25$, averaged to 1.00. The ranking of the geometric means of element legibilities is given in the last column of Table 1.

Cardinal-direction indications showed good legibility. They ranked second highest in legibility among the sign categories tested. The possibility was also considered that the superior legibility of cardinal-direction indications might be due to practice on the previously administered cardinal-direction charts. To test this possibility, an additional 10 subjects who had no previous practice on cardinal-direction indications were tested solely on sign replicas. The Spearman rank-difference correlation between the last column of Table 1 and the corresponding legibility rankings recorded for the new group was 0.88. On the additional tests, cardinal-direction indications had the best legibility of the eight sign elements, place names ranked fifth as indicated in Table 1, and route numbers had the poorest legibility--i.e., they again ranked eighth.

The finding that cardinal-direction indications do not pose a legibility problem is in contradiction to the letters of complaint received from several motorists. It is possible that directional signs may be confused if the driver is careless or if he or she views the signs under the adverse visibility conditions of rain, snow, or dusk. In addition, drivers may not be sure of the cardinal direction of their destination. However, it does not seem reasonable to insist that the cardinal-direction indications be even more legible than they currently seem to be. Until firm evidence accumulates that cardinal-direction indications pose a special reading problem, the matter of improving their legibility must be held in abeyance.

Route numbers had the lowest legibility of the sign elements tested (Table 1). The legibility distance of route numbers was 10.1 percent less than that of place names. The nine possible comparisons of place-name versus route-number legibilities show differences to be beyond the 0.01 chance level of significance (binomial expansion test).

The relatively poor legibility showing of route numbers may be due to several difficulties. Even if the driver recognizes that he or she is viewing a route number, there are a total of 90 possible two-digit numbers and 1000 three-digit numbers. In addition, route numbers are often crowded on the Interstate seal. In contrast, an information element such as "NEXT RIGHT" occurs in a characteristic position on the sign, is not cluttered by other information, and presents minimal possibilities of choice. The relatively uncluttered exit and distance numbers also showed better legibility than route numbers.

A number of clues for improving sign legibility were suggested by the test subjects' comments. The white-on-blue Interstate route number had poorer legibility than the black-on-white U.S. route number (sign 3). The Interstate route number appears more cluttered than the other. A number of subjects mentioned that the diagrammatic arrows of signs 3 and 4 indicating splits and interchange geometrics could be seen before the sign elements were readable. The excellent legibility of diagrammatic symbols may give a useful orientation to upcoming road geometry. On sign 4, the cardinal direction "East" is surrounded by abundant uncluttered space. This cardinal-direction indication had very good legibility. It is likely that many signs can be made more legible by the application of effective graphic design principles.

The guide-sign legibilities in Table 1 are expressed as visual angles in Table 2. Table 2 indicates the relative threshold size of the sign elements at a fixed distance. For example, to be seen at the same distance as a place name, a route number on these signs would have to be, on the average, 1.17 times as tall. (The result is given by the division of the two geometric means, i.e., $3.941/3.358 = 1.174$.)

The angle figures given in Table 2 can also be used to determine legibility distances if the size of the element is known. Extrapolated from the 3.358-min visual angle threshold, a place name with 38.4-cm (15-in) high lower-case letters would have a legibility distance of 390 m (1200 ft). The legibility distance of a route number of the same size, calculated from the median visual angle of 3.94 min, would be 332 m (1090 ft). A cardinal-direction indication 38.4 cm in height would be seen at 476 m (1536 ft). Under these conditions of equal size, the route number would just be seen 58 m (190 ft) closer than the place name and the cardinal-direction indication 86 m (282 ft) farther away. The legibility results of Tables 1 and 2 need to be interpreted in relation to drivers' guidance needs. If drivers followed a stereotyped pattern in viewing a sign, the application would be straightforward. For example, if route numbers were always read first, followed by place names, cardinal-direction indications, distances, etc., the element legibilities could logically conform to the same sequence. Route numbers would be made most legible of the elements, followed by place names, and so on. Unfortunately, information on drivers' sequential reading of guide-sign elements is not available. Moreover, drivers' guidance needs differ. Some drivers may be looking for a route, others for a

**Table 2. Legibilities of guide-sign elements expressed as visual angles.**

| Element | Threshold Visual Angle (min) Sign 1 | Sign 2 | Sign 3 | Sign 4 | Geometric Mean | Rank |
|---|---|---|---|---|---|---|
| "EXIT" | 2.49, 2.40 | 2.07 | 2.26 | | 2.253 | 1 |
| Exit number | 3.95, 3.91 | 3.29 | 3.32 | | 3.501 | 7 |
| Cardinal direction | 3.23, 2.67 | 3.20, 2.94 | 2.82 | 2.24 | 2.750 | 4 |
| Place name | 4.04, 4.02 | 3.27, 2.94 | 3.30 | 3.08 | 3.358 | 6 |
| Route number | 3.71 | 4.70, 4.92 | 4.05, 4.53 | 3.15 | 3.941 | 8 |
| Distance number | | 2.34 | 2.42 | | 2.380 | 2 |
| "NEXT RIGHT" | 3.24 | | | 3.07 | 3.154 | 5 |
| "EXIT ONLY" | | | 2.62 | | 2.620 | 3 |

Note: Based on the median distances given in Table 1.

place name, and still others may require no information from a sign. Therefore, a completely satisfactory, fixed formula for assigning sign element legibilities is not apparent.

However, it seems safe to assume that place names and route numbers are of primary interest and should be very legible. Elements such as "NEXT RIGHT", "EXIT ONLY", and distance indications such as "2 MILES" are subsidiary. They have meaning only when the relevant place or route number has first been identified. These subordinate, descriptive items are presumably read at closer distances and hence may be made less legible. Route numbers should not be difficult to read. They should be more legible than they are on present guide signs.

## Comparison of Legibility of Block and CLC Letter Styles

The number of test subjects passing the cardinal-direction charts at each viewing distance is given below (1 m = 3.28 ft; N = 50):

| | Number Passing | |
|---|---|---|
| Distance (m) | Block | CLC |
| 12.19 | 1 | 1 |
| 10.67 | | 8 |
| 9.14 | 9 | 19 |
| 7.62 | 16 | 15 |
| 6.10 | 18 | 5 |
| 4.57 | 4 | 2 |
| 3.05 | 2 | |
| Total | 50 | 50 |

Visual angle was 2.59 min for median block style and 2.20 min for CLC style. The CLC chart is calculated to be read at an 18 percent greater distance. The legibility advantage of the CLC style is also shown by individual performance comparisons. Forty-five of the 50 subjects were able to read the CLC chart at a greater distance than the block chart. The remaining five subjects read both charts at the same distance. This difference would be obtained less than once in 100 trials if both charts were equally legible (binomial expansion test).

It will be recalled that the cardinal-direction indications were printed in the two styles on the sign replicas. Half the experimental group (25 subjects) viewed sign replicas with block letters, and the other half read the signs with CLC cardinal directions. The groups were closely equated in acuity. The median Snellen E score of the conventional group was 20/10.77, and that of the experimental group was 20/12.4. These scores did not differ significantly. Median distances at which the cardinal directions were read on the replicas are given in Table 3. On the average, the CLC indications were read 10 percent farther off than their block-letter counterparts. This result is in conformity with results obtained by Forbes and others (3) in their full-scale study. These investigators also found 10 percent greater legibility for CLC than for block-style lettering.

## Abilities Involved in Reading Guide Signs

The intercorrelations of the various experimental tests for sign 2 are indicated in Table 4. The elements of the sign are also identified. The statistical signs of Snellen test correlations have been reversed to compensate for the fact that a low Snellen score represents superior visual acuity. The first intercorrelation in the table (0.75) represents the Pearsonian product-moment correlation of Snellen scores with scores on the block-letter cardinal-direction chart. The other entries are similarly interpreted.

The intercorrelations of Table 4 imply that abilities other than visual acuity are involved in reading road signs. As Table 5 indicates, the acuity type tests--the Snellen E, block, and cardinal-direction charts--intercorrelated highly. Intercorrelations ranged from 0.75 to 0.95. The high correlation of 0.95 represents that between the two cardinal-direction charts. The various guide-sign elements also intercorrelated highly. The averages of each with all the others ranged from 0.793 to 0.876. These averages do not include the 1.00 correlation of each test with itself. In contrast, acuity tests and guide-sign elements show lower correlation with each other. The averaged correlations ranged from 0.529 to 0.670.

These results suggest a possible advantage to including a sign-reading test in the driver-licensing examination. The ability of drivers to read signs at a distance is only partly related to visual acuity. Persons of very low literacy or

**Table 3. Median legibility distances at which cardinal-direction indications were read on highway sign replicas.**

| | Sign 1 | | Sign 2 | | Sign 3 | Sign 4 |
|---|---|---|---|---|---|---|
| Item | South | West | North | East | North | East |
| Distance (m) | | | | | | |
| Block | 4.32 | 4.91 | 3.00 | 3.27 | 2.48 | 7.02 |
| CLC | 5.11 | 5.25 | 3.28 | 3.73 | 2.60 | 7.51 |
| Advantage of CLC over block (%) | 18 | 7 | 9 | 14 | 5 | 7 |

Note: N = 50.

**Table 4. Intercorrelations among acuity tests and guide-sign elements.**

| Variable | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 0.75 | 0.78 | 0.69 | 0.70 | 0.72 | 0.74 | 0.72 | 0.70 | 0.68 | 0.51 | 0.67 | 0.57 |
| 2 | | 0.95 | 0.50 | 0.57 | 0.56 | 0.62 | 0.49 | 0.46 | 0.51 | 0.44 | 0.53 | 0.61 |
| 3 | | | 0.55 | 0.65 | 0.60 | 0.70 | 0.52 | 0.53 | 0.53 | 0.46 | 0.56 | 0.63 |
| 4 | | | | 0.92 | 1.00 | 0.88 | 0.86 | 0.81 | 0.89 | 0.81 | 0.91 | 0.74 |
| 5 | | | | | 0.92 | 0.97 | 0.83 | 0.78 | 0.86 | 0.78 | 0.81 | 0.75 |
| 6 | | | | | | 0.90 | 0.86 | 0.80 | 0.89 | 0.83 | 0.91 | 0.77 |
| 7 | | | | | | | 0.82 | 0.79 | 0.86 | 0.80 | 0.81 | 0.76 |
| 8 | | | | | | | | 0.93 | 0.88 | 0.71 | 0.86 | 0.71 |
| 9 | | | | | | | | | 0.83 | 0.69 | 0.85 | 0.66 |
| 10 | | | | | | | | | | 0.77 | 0.86 | 0.74 |
| 11 | | | | | | | | | | | 0.82 | 0.84 |
| 12 | | | | | | | | | | | | 0.75 |

Notes: N = 50.
1 = Snellen E test, 2 = block-letter cardinal-direction chart, 3 = CLC cardinal-direction chart, 4 = "EXIT" legibility distance, 5 = "17" legibility distance, 6 = "North" legibility distance, 7 = "East" legibility distance, 8 = "270" legibility distance, 9 = "495" legibility distance, 10 = "Barstow" legibility distance, 11 = "Clearwater" legibility distance, 12 = "2" legibility distance, and 13 = "MILES" legibility distance.

Table 5. Principal intercorrelations and averages of intercorrelations.

| Intercorrelation of Variables | Correlation | Implied Comparison |
|---|---|---|
| 1 with 2 | 0.75 | Acuity tests among themselves |
| 1 with 3 | 0.78 | |
| 2 with 3 | 0.95 | |
| Average | | |
| 1 with 4-13 | 0.670 | Acuity tests versus sign elements |
| 2 with 4-13 | 0.529 | |
| 3 with 4-13 | 0.573 | |
| 4 with 5-13 | 0.869 | Guide-sign elements among themselves |
| 5 with 4-13 | 0.847 | |
| 6 with 4-13 | 0.876 | |
| 7 with 4-13 | 0.846 | |
| 8 with 4-13 | 0.829 | |
| 9 with 4-13 | 0.793 | |
| 10 with 4-13 | 0.842 | |
| 11 with 4-13 | 0.783 | |
| 12 with 4-13 | 0.842 | |

intelligence were not included in these tests. If such persons were involved, they would be expected to have particular difficulty in comprehending the sign elements.

## Walk-To Method of Testing Sign Legibility

The walk-to method of testing sign legibility was successfully used in this study. However, the testing positions become crowded at distances close to the charts. Position 2 was only 0.217 m (8.5 in) farther from the chart than position 1, which was itself only 1.08 m (3.5 ft) from the chart. At these close distances, it becomes difficult to precisely position a subject. Results are affected by slight forward leanings and swayings. It is recommended that testing be done at greater distances. Words with letter heights of 0.88 cm (0.348 in) are read by persons with 20/20 normal vision at about 6.1 m (20 ft). Persons with better than normal vision could read words with this letter height at distances ranging up to 12.2 m (40 ft) or more. These considerations imply that signs with a minimum letter height of 0.85 cm (0.33 in) would be satisfactory in this type of legibility testing.

Sign messages graded in size, such as those used on the Snellen charts, might also be used in legibility testing. However, it is not easy to reproduce the signs in a graded size series adjusted to testing a subject group. For most legibility investigations, the walk-to method of varying visual angle offers a most convenient and effective test method.

## SUMMARY

This study was concerned with the legibility of information displayed on highway guide signs, and particularly cardinal-direction indications (North, South, East, West). The work was carried out in the FHWA vision testing alley by using scaled-down replicas of highway signs. A Snellen E chart was also included in the testing as a check on visual acuity. The test subjects attempted to read the signs at successively decreasing distances until all elements were correctly interpreted. The data obtained in the experiments support the following conclusions:

1. In contrast to what was assumed at the start of the study, the cardinal-direction indications demonstrated satisfactory legibility. They could be identified at a greater distance than any of the sign elements except the message "NEXT RIGHT".
2. CLC cardinal-direction indications were seen 10 percent farther away than conventional block letters on the sign replicas. The use of CLC lettering can increase element legibility without requiring appreciably more sign space.
3. Of the guide-sign elements tested, route numbers had the poorest legibility. They were identified at 10.1 percent shorter distance than place names.
4. The analysis of test intercorrelations indicated that abilities other than visual acuity are involved in reading guide signs. The guide-sign elements intercorrelated more highly among themselves than they did with the acuity test. The possibility of including sign-reading acuity tests in driver-licensing examinations should be considered.
5. The study supports the feasibility of using small-scale replicas in performing research on sign legibility. A minimum letter height of 0.85 cm (0.33 in) is recommended to achieve adequate subject testing distance.

## REFERENCES

1. T.W. Forbes, ed. Human Factors in Highway Traffic Safety Research. Wiley, New York, 1972.
2. W.D. Zoethout. Physiological Optics, 4th ed. Professional Press, Chicago, 1947.
3. T.W. Forbes, K. Moskowitz, and G. Morgan. A Comparison of Lower Case and Capital Letters for Highway Signs. Proc., HRB, Vol. 30, 1950, pp. 355-373.
4. S.L. Smith. Letter Size and Legibility. Human Factors, Vol. 21, No. 6, 1979, pp. 661-671.
5. National Joint Committee on Uniform Traffic Control Devices. Manual on Uniform Traffic Control Devices for Streets and Highways. FHWA, 1971.

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# Roadway Visibility Using Minimum Energy

D.M. FINCH

The basic requirements for nighttime roadway visibility are reviewed, and a table of recommended values for roadway lighting of city streets is presented. The values are based on five classifications of streets and a separate category for intersections. The primary standard of measurement is roadway luminance, but illumination and glare values are shown as parallel requirements. The recommended values for roadway lighting are less than the current American Standard Practice (RP-8, 1978) values but have more stringent quality specifications. Thus, it is shown that vision on roadways can be equal to or better than current practice and energy use can be appreciably reduced, by as much as 50-60 percent in some cases. Data for a study project in Portland, Oregon, are reviewed to demonstrate that the recommendations are achievable and that energy and cost reductions are practical.

Roadway lighting systems have evolved over the years from gas lamps to arc lamps to incandescent lamps to mercury lamps and now to other types of gaseous discharge lamps. The available alternatives raise many questions and cause engineering and administrative dilemmas in selecting the most cost-effective system for new or retrofit projects. This paper attempts to identify some of the factors to be considered and to quantify some of the parameters that are necessary to achieve satisfactory roadway visibility with minimum energy consumption.

The cost of energy is paramount in the minds of most administrators of roadway lighting systems because of the escalations that have occurred in recent years and the projections for further increases in the future. When the life-cycle costs (the total annual costs) are analyzed, it is invariably found that energy costs are far greater than all other costs. The multiplier may be 4-10 times the capital and maintenance costs combined, depending on the type of system and the energy rate. So it is important to consider minimum energy systems.

This paper emphasizes city-street lighting systems rather than limited-access roadways such as freeways, thruways, and parkways. City-street lighting systems are frequently installed, maintained, and operated by public utility companies on a contract basis. The lighting is determined by a set of policy guidelines that prescribe the lighting on a design manual or recipe basis. Systems that are city owned and operated are usually established on similar bases. Reasonably satisfactory visual conditions are assumed to result when the guidelines are followed. The procedure is straightforward and simple, but the results are not always adequate in terms of either visibility or costs.

Many cities are faced with problems of future growth in both population and the provision of services, including vehicle and pedestrian traffic. Since the basic street pattern cannot be changed, it will be necessary to improve the operating efficiency of the present streets. This does not mean just "more of the same" but rather improved quality with less energy plus other visual aids and control devices that can increase traffic flow with safety.

City streets have different visibility requirements based on their use and functional demands. The following six categories apply to most city streets except those in the downtown central district:

1. Regional trafficway--Four or more lanes, without parking, 40-45 mph, mixed traffic with trucks, commercial vehicles, and passenger cars;
2. Major traffic, major transit--Four lanes, with parking, 30-35 mph, mixed traffic plus buses and pedestrians;
3. Neighborhood collector, major transit--Two lanes, with parking, 30-35 mph, mixed traffic plus buses and pedestrians;
4. Neighborhood collector, minor transit--Two lanes, with parking, 30-35 mph, few buses;
5. Local service--Two lanes, with parking, residential or local collector; and
6. Intersections.

These classifications are approximately parallel with the types of roadways used by the American Standard Practice for Roadway Lighting (RP-8, 1977). A new American Standard is to be issued in the near future in which there may be new names for the types of streets but the functions will remain the same.

The existing American Standard Practice (RP-8) does not address the problem of energy use in any way. No mention is made of the use of high-efficacy sources or the visual aspects of the roadway. The new proposed standard will attempt to remedy these omissions but, since it is not yet available, some of the important factors are reviewed here.

## TECHNIQUES TO PROVIDE GOOD VISIBILITY

Lighting codes or standards vary from one locality to another, depending on the local conditions and the requirements for critical seeing. Nevertheless, there are basic concepts and principles that are common to all nighttime traffic situations. The basic visual parameters that must be covered are (a) road surface luminance, (b) roadway illumination (required to produce luminance), (c) glare restrictions, and (d) optical guidance information.

The primary function of the lighting on roadways is to facilitate the movement of traffic, both vehicle and pedestrian. The lighting is considered to be of good quality if it ensures ease of perception, develops adequate adaptation luminance, is comfortable, and provides guidance information.

The driver's visual task is extremely complex and has not been synthesized or modeled completely. But several aspects of the task can be identified: (a) The driver must use advance planning for the run of the road ahead and must therefore have visual information on the alignment of the road, curves, dips, rises, intersections, and so on; (b) the driver must make short-term decisions on rate of closure with other vehicles, objects, and pedestrians on the roadway ahead; and (c) the driver must use short-term memory and mental data processing to keep track of his or her position, the position of other vehicles, and objects in the immediate surroundings.

Good visual conditions must hold over the whole of the roadway for some distance ahead and within the driver's immediate environment. Perception should be quick, reliable, and easy so that the visual tasks can be continued over a period of time without excessive strain and fatigue. Thus, ease and comfort in seeing are not luxuries that can be dispensed with.

The perception of relevant objects leads to action by a driver. The action may involve (a) proceeding with caution or no action, (b) decelerating or stopping, or (c) changing lateral position and turning. Among the relevant visual objects are (a) information objects, such as road markings, traffic lights, direction signs, and destination signs; (b) hazard objects, such as pedestrians, animals, parked cars, boxes, rocks, road damage, and so on; and (c) other moving vehicles on the roadway in the traffic stream.

Visual perception depends on (a) the luminances in the visual field; (b) the state of adaptation of the observer; (c) the size, shape, color, and pattern of objects; (d) motion; (e) time available; (f) the physiology of the observer's visual, neural, and mental systems; (g) age; and (h) many psychological factors, such as attention, familiarity, diversions, and so on. From the engineering point of view, one can prescribe the physical parameters for a design level of perception for a normal, attentive observer in a given age group. However, even though such a physical array will provide the environment for adequate visual perception, it will not ensure the perception of critical relevant objects due to the vagaries of human responses.

Luminance is the physical parameter that must be prescribed as the basic quantity in the visual field. Luminance is the luminous flux within a small solid angle that is incident on the observer's eye from a specific direction. The physical quantity, luminance, may cause a reaction within the observer's visual system that results in a subjective response termed "brightness". The brightness sensation is related to the external luminance, but it is not directly proportional since there are many nonlinear links in the perception system.

We have, therefore, a man-machine-environment system with both objective (physical) and subjective (human-factors) attributes. Only the objective specifications that relate to roadway lighting on city streets are developed in this paper.

Both quality and quantity considerations are needed in specifications for roadway lighting. Quality is measured by the uniformity of roadway luminance and illumination and by the relative freedom from glare. Quantity is measured by the average roadway luminance ($L_{ave}$) and illumination ($E_{ave}$). The primary measurement should be roadway luminance, since this is the parameter that directly affects visual perception. However, it is not enough to specify only luminance, since there may be unique situations in which the design procedure for luminance indicates a satisfactory pattern but the actual installation may not result in a satisfactory job. This may happen because of the limitations of the luminance calculation procedure, which assumes the observer to be at a fixed location and the roadway to be represented by one of the standardized reflection factor tables, neither of which is true.

Hence, in addition to luminance values, a set of illumination values should be prescribed. The rationale for this is that the collective experience of street-lighting designers in the United States relates to horizontal illumination values (footcandles) and thus the practitioners will be comfortable with a specification that requires known quantities. In addition, in order to develop roadway luminance patterns, it is necessary to have incident illumination on the street. There is not a one-to-one relation between luminance and illumination, but experience indicates that, for asphaltic road surfaces with twin-beam luminaires on about 25- to 40-ft mounting heights, there is an approximate correlation between $L_{ave}$ and $E_{ave}$ in which $L_{ave} \approx k \times E_{ave}$, where $0.25 \geq k \geq 0.15$ (1-4), when L is in footlamberts and E is in footcandles. Therefore, a given level of average luminance will have a requirement for average horizontal illumination.

The average roadway luminance and the nearby surroundings determine the visual adaptation level of the observer's eyes, which is related to visual performance criteria (e.g., the higher the adaptation level, the better the visual performance) (6).

In terms of seeing details on the roadway, the uniformity of luminance is very important. For example, a small object in an area of minimum luminance may not be visible, whereas the same object in another area of higher luminance may be easily perceptible. Three measures of uniformity are needed: (a) a measure of the minimum value on the roadway compared with the adaptation luminance ($L_{min}/L_{adapt}$), (b) a measure of the total luminance variation along the projected path of the vehicle [the latter given by the ratio of $L_{max}/L_{min}$ (longitudinal) along a line ahead of the observer], and (c) the ratio of $L_{max}/L_{min}$ (overall).

Another factor that must be specified in a roadway lighting system is the degree of glare that will be permitted. This is a subjective parameter that has been dealt with in different ways by different researchers and regulatory bodies. The effect is caused by high-brightness sources in the field of view that cause a reduction in the ability to see objects and cause discomfort, fatigue, and annoying responses. The current state of the art tries to isolate the disabling aspect from the discomfort aspect without complete success. Research studies have established the physical parameters that are significant in glare evaluation. They are (a) luminance of the source, (b) location in the field of view, (c) size of the source, (d) adaptation luminance, and (e) age of the observer. The combination of multiple sources is reported to be nonlinear insofar as comfort is concerned and approximately linear insofar as disability is concerned.

With the above brief description of the parameters required for good roadway visibility and by reference to selected research (1-3, 6-12), it is possible to establish recommended values for illumination, luminance, and glare. The recommended values are given in Table 1. The values for illumination are slightly lower than in the current American Standard Practice, but they are fully justified on the basis of the improved quality provided by the luminance and glare requirements, which provide for adequate visibility. Their application can provide energy-efficient systems. Present connected loads can be reduced as much as 50 percent in many areas where these recommendations are applied by using high-efficacy sources and luminaires.

## RECOMMENDED LIGHTING STANDARDS

### Explanation of Values

Before the bases of the recommended standards are discussed, specifications for the use of the measurements given in Table 1 are explained.

#### Horizontal Illumination

The value of average horizontal illumination [$E_{h(ave)}$], in footcandles, is calculated as the average over the area of the traffic lanes including the center median and bicycle lanes, if any. The area for $E_{h(ave)}$ does not include parking lanes, sidewalks, berms, or other areas outside of the traffic lanes. A parking lane is assigned 7 ft of width, to be subtracted from the curb-to-curb width. For design calculations, the end-of-life lamp lumens are used together with an appropriate luminaire maintenance factor. Areas out to 15 ft to each side of the outside traffic lane shall be lighted to $\geq 0.2$ footcandles (average) if such areas are used for parking or pedestrian traffic.

#### Luminance

$L_{ave}$, measured in footlamberts, is the average luminance within the traffic lanes from a transverse

**Table 1. Recommended street lighting standard.**

| Street Classification | Horizontal Illumination (footcandles) | | | Luminance (footlamberts) | | | | Glare | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | $E_{h(ave)}$ | Ave/min | Max/min | $L_{ave}$ | Ave/min (overall) | Max/min (overall) | Longitudinal | GM | TI | $L_v$ |
| Regional trafficway | ≥1.2 | ≤3 | ≤9 | ≥0.30 | ≤2.5 | ≤5 | ≤2.5 | ≥6 | ≤20 | ≤0.05 |
| Major traffic, major transit | ≥1.0 | ≤3 | ≤9 | ≥0.24 | ≤2.5 | ≤5 | ≤2.5 | ≥6 | ≤20 | ≤0.04 |
| Neighborhood collector | | | | | | | | | | |
| Major transit | ≥0.7 | ≤3 | ≤9 | ≥0.18 | ≤2.5 | ≤5 | ≤2.5 | ≥6 | ≤20 | ≤0.03 |
| Minor transit | ≥0.5 | ≤3 | ≤9 | ≥0.12 | ≤2.5 | ≤5 | ≤2.5 | ≥6 | ≤20 | ≤0.02 |
| Local service | ≥0.2 | ≤6 | ≤20 | ≥0.06 | ≤5 | ≤10 | None | ≥4 | ≤25 | ≤0.05 |
| Intersections | | | | $L_{ave(i)} \geq 1.5\ L_{ave(r)}$<br>Unif. $[L_{ave(i)}/L_{min(i)}] \leq 2.5$ | | | | ≥7 | ≤10 | ≤0.05 |

line approximately 100 ft ahead to about 400 ft ahead of the observation point. The lateral boundaries shall include the area of the traffic lanes. At least 20 points shall be used to calculate $L_{ave}$, at least 5 points along the centerline of the lane ahead of the observation point. The individual luminance points shall be calculated or measured from a point 4.5 ft above the roadway located approximately in the center of the outside lane and at longitudinal points located to include the maximum longitudinal variations in road luminance. For roadways with two-way traffic, the luminances shall be determined for each direction of traffic if the luminance pattern is asymetric.

Field measurements will be made with a suitable telephotometer that uses an acceptance aperture with a 2-arc-minute vertical angle. At least 20 points will be measured on the roadway within the prescribed area at approximately equal angular increments. The $L_{ave}/L_{min}$ ratios (longitudinal and overall) shall be calculated for each observer location and shall consider all of the luminances within the area. The $L_{max}/L_{min}$ ratios shall be calculated overall and along the centerline of the outside lane for each direction of traffic and shall be met for observer locations.

## Glare

Glare is evaluated by two criteria: (a) discomfort glare and (b) disability glare.

### *Discomfort Glare*

The discomfort from glare is described by a glare control mark (GM) (4,8), which expresses on an ordinal scale the subjective appraisal of the degree of discomfort experienced. The value of GM is related to different glare sensations as follows:

| GM | Glare Sensation |
|---|---|
| GM-1 | Unbearable |
| GM-2 | Disturbing |
| GM-5 | Just admissible |
| GM-7 | Satisfactory restriction |
| GM-9 | Unnoticeable |

These words are not intended to indicate an absolute level of glare. They are listed here as used in experiments of the International Commission on Illumination (CIE) (1,4,8). The subjective appraisal of the glare and the associated value of the GM depend on the photometric and geometric characteristics of the lighting installation, and the GM should be used as a relative index.

### *Disability Glare*

The method of evaluating disability glare is based on the Holladay formula (7). According to the formula, the effect of glare is quantified by an equivalent uniform luminance that describes the effect of the stray light in the eye--lowering the contrast. The relative threshold increment (TI) is expressed as the difference between the threshold under glare conditions and its value without glare, expressed as the percentage of the value without glare. The veiling luminance ($L_v$) represents the illumination at the eye due to glare sources and is the equivalent uniform luminance, in footlamberts, superimposed over the entire visual field. To evaluate its effect, this value can be compared with the average luminance or the adaptation luminance.

### *Recommendations on Glare*

The recommendations in Table 1 concerning the restriction of glare in road lighting installations have been given in terms of GM and TI. These values should be considered as minimum requirements. If higher values for GM and lower values for TI are economically feasible, preference should be given to such improvements.

Field measurements of glare should be made by using a telephotometer located at the luminance observation location. The photometer should use a 6-arc-minute aperture (a 2-in circle at 100 ft) and should have a mount that can give the vertical and horizontal angles of the photometer axis with respect to a reference line of sight. All sources within the normal field of view of a driver that are greater than 20 times the average road luminance should be measured for maximum luminance. The approximate field of view to be covered should be ±30° horizontal and +20° to -5° vertical. The location and magnitude of each glare source should be recorded. If the sources subtend a solid angle greater than 0.0002 steradians (2 ft² at 100 ft), separate measurements should be made to give the average luminance.

## Intersections

The area used to determine $L_{ave}$ is the roadway area within the traveled lanes extending from the approximate centroid of the intersection along each lane to a transverse line 10 ft beyond the point of entry. $L_{ave(i)}$ is the average luminance in the intersection, $L_{ave(r)}$ is the average luminance of the intersecting road with the highest value, and $L_{min(i)}$ is the minimum luminance within the intersection.

## Bases for Recommended Standards

As discussed below, the recommended lighting standards are based on street characteristics, user requirements, and experience with comparable lighting applications.

## Street Classifications

### *Regional Trafficways*

The regional trafficways classification (class 1) represents the greatest demand for visibility because of the relatively high speed of the traffic (45 mph), the high volume (up to 25 000 vehicles/day), and the mixture of types of vehicles (trucks, commercial vehicles, buses, and passenger vehicles). Experience on such streets has shown that average illumination levels on the order of 1.2 footcandles and average luminance levels on the order of 0.3 footlambert are satisfactory when accident records and traffic flow are used as criteria (1,2,6,10-12). In similar traffic conditions in Europe and Japan, the average roadway luminance is specified as 1 cd/m² (0.29 footlambert) and in Canada as 1.2-0.8 cd/m² (0.35-0.23 footlambert) (1-3). The British use a combination of illumination and luminance specifications (13,14).

In a study (14) that covered a total of 38 installations, wet and dry, the range of $L_{ave}$ was from 0.05 to 0.38 footlambert. The roads are described as ordinary traffic routes and motorways. These levels have been in use for some time and have been accepted as adequate. The Canadian values (3), which are approximately the same as those in Table 1, are being seriously considered as the basis for new U.S. values (i.e., American National Standards Institute/Illuminating Engineering Society) to be issued in the near future.

The recommended average illumination values in Table 1 are less than those currently recommended by IES for similar street classifications (RP-8). The lower values are suggested for several reasons: (a) Their extensive use in other countries has demonstrated their adequacy, (b) a ratio of maximum/minimum is specified to ensure better-quality lighting, and (c) luminance and glare restrictions are added in order to improve the quality and comfort aspects. Thus, it has been demonstrated in the reference articles that, with an improved quality of lighting, a reduced quantity of light can be adequate for safe operations and may improve the existing visual environment and will save energy.

Most authorities recognize the importance of roadway luminance and the deleterious effect of glare. In the past, the problems of their specification related to the methods by which the luminances were determined and the relative importance of the glare parameters. The problems are now solvable. The quantities, definitions, and techniques can now be described, measured, and calculated by use of currently available instruments and computer programs.

The luminance calculations for a roadway are based on the candlepower distribution of the luminaires and the R-3 classification of road surface reflection. R-3 is a CIE designation for asphaltic concrete with gravel sizes up to 10 mm but harsh texture (4). This may not truly represent all city streets, but in lieu of better data it is reasonably adequate. Luminance measurements on many city streets indicate an overall ratio of $L_{ave}/E_{ave} \approx 0.25$ to 0.15. The CIE data show a ratio of ≈0.17.

The glare calculations are based on CIE recommended techniques for evaluation of comfort and disability (8). These formulas have been criticized by several researchers as lacking correlation with subjective evaluations. The disagreements deal mostly with the details of the coefficients, the exponents used in the equations, the size of the visual field, and the adaptation luminance of the observer. Thus, while research continues and the techniques are being refined, the recommendations in this paper use the CIE recommended procedures and the established computer programs.

The fact that weather causes streets to be wet part of the time is another factor in establishing the suggested values of E, L, and glare. The values listed are for dry roadways, but the combination of ratios of $E_{ave}/E_{min}$, $E_{max}/E_{min}$, $L_{ave}/L_{min}$, and $L_{max}/L_{min}$ ensure that under wet conditions the roadway luminance pattern will not deteriorate severely.

### *Local-Service Streets*

At the other end of the lighting spectrum are the local-service roads (class 5). As used in this paper, a local street is a two-lane road with parking on one or more sides. The street has minor traffic and essentially serves as a collector. Daily traffic volume may vary from a few vehicles to approximately 2000. There is usually parking along at least one curb, and there may be pedestrians in crosswalks and on sidewalks. The block lengths vary from short (about 200 ft) to normal (about 400-500 ft). Many of the streets are in a rectangular grid pattern of blocks. Ideally, such streets should be lighted to minimum standards by use of a continuous lighting system. These streets are differentiated from purely residential streets where the intersections constitute the principal traffic-control and conflict areas.

The basis for the minimum luminance specification is that at any point the minimum level on the roadway should be at least equivalent to full moonlight (approximately 0.01 footlambert). This is a consensus value that has been used for many years and is based on experience and public acceptance. This is the order of magnitude of the average roadway luminance produced by low-beam headlamps on an asphaltic roadway. The average should be 5 times the minimum--i.e., ≥0.05 footlambert--and the maximum 10 times the minimum--i.e., ≥0.10 footlambert. This corresponds to an average illumination of E ≈ 0.2 footcandle, where the luminance is 0.05 $L_{ave}$. It is assumed that the lighting is continuous along the street, spacings are about 8 times the mounting height, and the intersections are lighted to ≈1.5 times the average roadway luminance between intersections. From a practical point of view, this means that for 400-ft blocks there should be at least one intermediate light but for 200-ft blocks there may be lights only at the intersections.

The 0.20-footcandle (0.05-footlambert) level should be the minimum average level acceptable for continuously lighted local-service streets. There may be (and probably are) other streets that are not eligible to be, or are not warranted to be, continuously lighted. If these streets have regions of potential traffic conflict that should be lighted, such areas should be considered as intersections and lighted accordingly.

In most instances, moving vehicles will be operating on local-service streets with low-beam headlights. For dry-road conditions on asphaltic surfaces, the maximum foreground luminance due to headlights on the roadway will be about 0.10 footlambert and the average will be about 0.05 (15). This means that the driver's adaptation level will remain about the same on both lighted and unlighted local-service streets. Glare considerations may have to be relaxed because of the long spacings. The prevalence of trees in such areas may alleviate some of the deleterious effects of glare. In any case, the vehicle speeds are usually low, which will to some extent compensate for the higher glare levels. Table 1 permits higher glare values for local-service streets.

The value of $E_{ave}$ for local-service streets is the same as that recommended by the American Standard Practice of 1964 (16). Since that time, the recommended average value of 0.2 footcandle was doubled to 0.4. There is no rationale or apparent reason for this change. The American Standard Practice currently recommends 0.2 footcandle for alleys and sidewalks (most local-service streets have sidewalks). The suggested average illumination in Table 1 for local-service streets is also 0.2 footcandle.

Thus, it is logical to prescribe an average of 0.2 footcandle for the roadway, parking area, and sidewalk in a local-service area. Such a specification ensures some spill light that will provide additional guidance information, security, and visual enhancement of trees, yards, and house fronts.

### *Intermediate Types*

The following street classifications represent intermediate types between regional trafficways and local-service streets and are based on traffic volume, speed, parking, mixes of vehicle types, loading and unloading of passengers and goods, pedestrian crossings, and sidewalk activities. Each needs varying amounts of light based on the user's visibility requirements. Values for class 2, 3, and 4 streets are logical intermediate steps between class 1 and 5 streets, based on driver, pedestrian, and other use demands.

1. Major traffic, major transit (class 2)--Class 2 streets have design speeds in the 35- to 40-mph range (many vehicles travel faster) and traffic volumes up to 20 000 vehicles/day. These streets should therefore have almost as much light as regional trafficways. There are usually businesses along these routes that contribute additional light to the roadway. Sidewalks, driveways, and loading zones are usually located around commercial establishments or near intersections where additional lighting is available.
2. Neighborhood collector, major transit (class 3)--Class 3 streets have somewhat fewer vehicles than class 2 streets (up to 10 000 vehicles/day) but have a large number of buses with loading and unloading requirements and with associated pedestrian traffic. Therefore, the lighting requirements during commuting hours will be fairly high. The suggested levels are lower than those for class 2 streets, but they are not absolute, since there are no clearly defined boundaries between the classifications. The lighting levels may be adjusted to the demand requirements (e.g., it may be desirable to reduce the lighting during hours when the traffic is light, such as from midnight to 5:00 a.m.).
3. Neighborhood collector, minor transit (class 4)--Class 4 streets generally have traffic volumes in the range of 2000-6000 vehicles/day, including some, but not many, buses. Such streets are relatively important traffic routes and have greater visibility requirements than local-service streets, though not as great as class 3 streets. Here, too, the level can be adjusted to meet the demand requirements and could be reduced between midnight and 5:00 a.m., if necessary, for energy conservation.

## Additional Bases for Recommendations

In an extensive review of street lighting in relation to road safety, Fisher (6) makes the following points that are relevant to our recommendations:

1. There is a good deal of agreement among countries on street lighting requirements. Most countries have a multilevel requirement related to the importance of the roadway (and presumably the difficulty of the visual task) and recommend an average roadway luminance of 1-2 cd/m² (0.3-0.6 footlambert) for the most important roads. Australia has a single minimum standard for ordinary urban traffic routes that is equivalent to a roadway luminance of about 0.75 cd/m² (0.22 footlambert). With these levels, the reduction in night accidents should be about the maximum that can be obtained through lighting alone.
2. For the case of a lighted road with an absence of vehicle lights, there is maximum probability of detecting pedestrianlike objects at a luminance level of 1 cd/m² (0.3 footlambert) (17).
3. Data obtained by Blackwell suggest that modest lighting levels of around 1 cd/m² result in relatively high levels of visual performance (18).
4. As shown by Narisada (9), visual performance, in terms of the probability of detecting a small standard object seen against the roadway, is related to both the light level and its uniformity. The light level can be lowered without adverse effect, provided uniformity is upgraded at the same time.
5. A comparison of the lighting codes of 16 nations (10) (excluding Australia) showed a common feature--i.e., a multilevel lighting requirement covering various classes of roads. The more heavily trafficked the road, the higher was the requirement. In addition, the requirements for the most important traffic routes were found to be similar: Most values were between 1 and 2 cd/m² (0.3-0.6 footlambert).
6. The material reviewed provides good evidence to suggest that the minimum luminance level to be used on urban traffic routes that have a mixed road-user population should be similar to the Australian minimum standard of 0.75 cd/m² (0.22 footlambert).

## Impact of Recommended Standards

### Energy Savings

The recommended standards can be met by using several types, sizes, and spectral distributions of light sources [e.g., Mercury (Hg), metal-halide (MH), high-pressure sodium (HPS), or low-pressure sodium (LPS)] in various types of luminaires made by a number of different manufacturers.

On roadways now using 1000-W Hg sources, the requirements can be met by using 400-W HPS lamps and core-coil ballasts or 310-W HPS lamps and solid-state ballasts. The energy savings are not directly proportional to lamp wattage, but it is reasonable to estimate that the power used can be on the order of half the present load. Similarly, on the other classes of roads that now use 400-, 250-, or 175-W Hg lamps, the power can be reduced by 50-60 percent. Then, if switching circuits are used on some of the roads to turn off or reduce the output of part of the system during off-peak hours (e.g., from midnight to 5:00 a.m.), an additional fraction (approximately 23 percent) of the power used on the road can be saved. This calculation is based on the assumption that the street lights are now on for 4000 h/year or about 11 h/day.

Thus, the energy consumption for a major road, such as Sandy Boulevard in Portland, Oregon (19), could be reduced from the present ≈95 700 kW·h/mile/year to 59 300 kW·h/mile/year, which at the low Portland rate of \$0.03/kW·h would be a saving of about \$1092/mile/year. Other streets in Portland (19) could have about the same percentage reduction; this yields an average cost reduction of about \$700/

mile/year. Thus, if the estimated 1500 miles of Portland city streets were brought to the recommended quantity and quality levels by using modern efficient light sources, there would be an estimated potential savings (based on an energy cost of \$0.03/kW•h) of about 1500 miles x \$700/mile = \$1 000 000/year.

### Traffic Safety

Traffic safety is a less tangible quantity than energy savings. The supporting data on which to base the recommended lighting levels indicate that traffic safety increases with lighting level up to an inflection value (18). The relation is not linear. There are rapid gains in safety for increases in low lighting levels, but a point of diminishing returns is reached as the levels are increased. A practical upper limit is in the range of 1-2 footcandle illumination or 0.30- to 0.60-footlambert luminance. Traffic safety is considered to be a function of traffic volume, time of day, speed, weather, and the state of alertness of drivers and pedestrians, among the important variables. All of these variables are aided by good roadway lighting. The hours of darkness are known to be more hazardous for vehicle operations. When all known factors except darkness are isolated and eliminated from accident records, reduced visibility at night is shown to be a prime causal factor in vehicle accidents.

### Pedestrian Safety

Pedestrian safety is a matter of prime concern on city streets (12). Accident records show more pedestrian-vehicle accidents in cities than any other type. Again, increases in the lighting levels at the lower end of the scale are more significant in improving the visibility of pedestrians than increases at the higher levels. The fundamental seeing problem is one of developing contrast between the pedestrian and the background. In most situations, the pedestrian will be perceived in silhouette as a dark object against a lighter background. If the roadway has a reasonably uniform luminance pattern, even though at a low level, a dark object can be perceived, especially if motion is involved. Headlamps help for objects in the foreground in such areas, but fixed lighting ensures that pedestrian safety will be improved (15,17).

### Adjacent Property Security

Adjacent property is always lighted to some extent by street lighting. This has both good and bad points. The good points are the following:

1. The light that does not fall on the traveled portion of the roadway will illuminate the parking areas, the sidewalks, the building facades (if any), and the yards and porches that may be adjacent to the road.
2. The spill light will broaden the visual field and raise the adaptation level of the driver.
3. Visual guidance and orientation information for the driver is increased.
4. A feeling of security and well-being is induced in pedestrians and residents along the street by a reasonable amount of spill light.
5. Street crime is generally lower on lighted streets and sidewalks than on unlighted streets (20).
6. Civic pride is enhanced, and this results in improved maintenance and cleanliness in lighted areas.

The bad points have to do with light trespass and atmospheric light pollution. There are situations in which spill light causes problems by shining in windows or on objects that should remain dark, such as certain plants that require a diurnal cycle of light and dark. Other special problems, such as insect control, discomfort glare, or claims of invasion of privacy, may also develop. Furthermore, in some localities spill light is of great concern to astronomers, both professional and amateur (21).

On balance, light on adjacent property in the amounts that may be produced by residential street lights is welcomed. In commercial, industrial, and retail business areas, the spill light may be requested by businessmen to aid in accenting business activities and to provide security lighting.

### Visibility

Visibility as applied to roadway lighting has a subjective meaning that is generally associated with the perception, recognition, and reaction to the visual scene and the details within the scene. The specification of visibility as a measurable quantity has been a goal of researchers and engineers for many years, but as yet it has not been satisfactorily quantified. Much of the work has been related to threshold values and to indicate how far above threshold a particular object might be. A number of thresholds can be used, such as size of object, time for observation, motion and color of object, and luminance contrast. Generally, the contrast and size thresholds are most commonly used as a visibility metric (7).

Several field instruments have been devised that can be used to reduce the contrast of a given object to threshold without changing the adaptation level of the observer. These instruments are useful for special-purpose tests and research, but they have not gained wide acceptance for field measurements. Basically, there is too much variation in the subjective measurements (22-24).

Another approach to visibility measurement and specification is that used in England, called the "revealing power" of the lighting system (13). This is a statistical approach in which the percentage of locations where a pedestrian can be seen is determined. Conditions are specified regarding reflectances, size, time, background luminance, and so on. The higher the revealing power, the better is the system. This system has been available for many years, but it has never gained wide acceptance outside England.

The work of Blackwell in the United States has led to yet another way of quantifying visibility. This is the "visibility level" (VL) method (24) or the "visibility index" (VI) method (25), which is a spinoff. The basis for both is the idea that at a given adaptation level the eye has a specific threshold contrast sensitivity. Laboratory research data on contrast detection are used as the basis for comparing field objects with the reference data. Visibility levels are then established that relate to how far above threshold the objects are in the given visual environment. The concept is good, but the fundamentals have yet to be translated into a workable index of roadway visual performance.

An attempt to adapt the VL system to roadways was made by the Federal Highway Administration (FHWA) through a research contract with the Franklin Institute (25). This work resulted in the VI method, which does calculate a number that is intended to be a measure of "visibility" on a roadway. There are many problems with the technique. One has to do with the fundamental definition of contrast. Other problems relate to the shape, size, and reflectance of the standardized target, the method of specifying the background luminance, the computation of roadway luminance, and so on. All of these indeterminate factors, plus the lack of experience with the method and the requirement for a computer to make the calculations, have retarded the use of this system.

The program for the VI method is available and can be used for the evaluation of selected streets. The VI values can be used in the overall selection of systems, but they are not a part of the recommendations given in Table 1. The VI levels are normally the average values for the roadway. A $VI_{15}$ value indicates the level above which 85 percent of the points on the roadway will lie. It may be noted that the developers of the system state that a VI of 1.50 represents their standard target in moonlight with a perception-reaction time of about 1.5 s for a driver going 30 mph. No recommendations are given for specifying VI levels.

## Application of Lighting Standards

### Types of Roadway Lighting

Most roadway lighting standards are focused on the lighting of continuous stretches of straight, uninterrupted roadways. Walkways, bikeways, natural or delineated pathways, sharp changes in route direction, intersections, diverging and merging areas, fixed hazards, and destinations are given little attention. Destinations in this case would include rest, waiting, and parking areas; bus stops; entries; exits; toll gates; terminals; and so on. "Curb-to-keyhole" security lighting has been almost totally ignored, although it is usually assumed that conventional roadway lighting will provide some security lighting.

### Emphasis Lighting of Critical Traffic Areas

The main point to be made here is that proper design of the lighting for critical areas has been largely ignored by codes and standards. But an excellent way to save traffic-lighting energy is to put that lighting where it is most needed in these critical areas and use less lighting in less critical areas.

Freeway lighting is a very specialized case of critical-area roadway lighting. It is worth noting that the State of California has long emphasized partial interchange lighting as opposed to continuous freeway lighting. Partial interchange lighting consists of lighting at critical traffic areas (off-ramps, ramp connections to crossroads, on-ramps, and merging and diverging areas). The lighting of city streets, of course, involves different problems.

But the lighting of critical traffic areas (mainly intersections) is more important than lighting roadways between intersections, especially in local-service areas. The greatest number of traffic events occur at intersections, and this is where lighting should have the greatest visual effectiveness. The lighting of critical areas calls for accent lighting--lighting with a strong visual attention effect (1,12).

Luminaire layouts for accent lighting should fit the geometry of the site. Generally, the uniformity of lighting within the accent area is much better than along a continuous ribbon of roadway. However, it is more difficult to specify and calculate average illumination for an area that is not as geometrically simple as a ribbon of roadway (streetside coefficient of utilization figures apply only to continuous ribbons of roadway). The basic rule for accent lighting layouts is to place a luminaire just beyond the traffic conflict area from the point of view of an approaching driver or pedestrian. At a simple cross intersection of two wide roadways, for example, a luminaire should normally be placed at the far right corner from the point of view of each approach direction (four luminaires). The design of a lighting layout for more complex intersection geometry is more difficult, but it still involves the application of the basic rule given above.

In order to achieve major savings in the energy consumed by city street lighting, the lighting of critical traffic areas should be considered of first importance. Low levels of illumination are not adequate for critical traffic areas, even though they might be sufficient for roadway delineation lighting and curb-to-keyhole security lighting. Since critical traffic areas are relatively small areas, the amount of power consumed in the lighting of these areas is likewise relatively small. If the same levels of illumination are specified for continuous roadways as for critical traffic areas, then the total power required goes up tremendously.

The lighting of "downtown" roadways between intersections is a special case. Higher levels of illumination are generally prescribed, not so much for traffic safety as to create a sense of security and to develop a phototropic effect that helps to attract people to business centers. But in outlying commercial, industrial, and residential areas, there is no need to have high levels of illumination between intersections. In the case of residential areas, minimum lighting at moonlight levels can be adequate. As a matter of fact, the American Standard Practice recommendation for walkway lighting currently specifies a minimum illumination level of 0.02 footcandle. As we know, the lighting of the moon, at about 0.02-0.05 footcandle, provides reasonable navigational night lighting; the only problem is that it is not always there. The energy-saving approach to city street lighting emphasizes the need for lighting at intersections and other critical traffic areas and implies that lower light levels can provide adequate visibility on roadways between intersections. The approach maximizes energy savings without compromising safety. Improved visibility at traffic conflict areas should improve safety; it certainly will not decrease safety! On the other hand, reducing light levels everywhere in order to save energy will reduce safety.

Because of the relatively close luminaire spacing required in traffic conflict areas, it is reasonable to use sharp-cutoff luminaires in these locations. Cutoff luminaires can improve visibility by enhancing visual attention to roadway objects and by reducing glare. If cutoff luminaires are used at intersections, noncutoff luminaires may be used between intersections. Then the possibility of confusing intersection lights with others will be minimized, and there will be increased contrast between intersection areas and the street areas between intersections. Such a layout would provide a desirable visual attention effect for critical traffic areas.

The use of sharp-cutoff luminaires at intersections and noncutoff units between intersections is given as only one example of how to achieve effective accent lighting for traffic conflict areas and relatively low-output diffuse lighting for intermediate roadway areas and for curb-to-keyhole security lighting.

# ENERGY-SAVING CONVERSIONS

## Street Classifications

In the following paragraphs, the recommended lighting standards are reviewed for each of the street classifications in the Portland, Oregon, study (19). Summary data for typical Portland streets are given in Table 2.

### Class 1: Major Traffic, Regional Transit

The present system of 1000-W Hg lamps in refractor-type luminaires approximately meets the recommended lighting standard for regional-type roadways (Table 1). The glare values are generally high and the uniformity ratios are not entirely satisfactory, but the system is reasonably acceptable except for the energy consumption. The circuit wiring and controls, including the photocell actuators, appear to be in good working order. The mast arms and poles can be used "as is" for the present luminaires or for conversion to a new set of luminaires. If a new system is installed, it will probably be a 400-W HPS with core-coil ballast or a 310-W HPS with solid-state ballast system, so the present wiring controls and auxiliary equipment can be used with only minor changes. If a switching circuit is desired so that some lights can be selectively turned off and on, a modification in the wiring will be required.

The recommended lighting values can be met by using (a) 400-W HPS lamps on standard ballast or (b) 310-W HPS lamps on electronic-controlled ballasts, in suitable, commercially available fixtures. This assumes the use of existing poles, a mounting height of approximately 40 ft, and 150-ft average pole spacing along one side of the roadway.

Cost calculations for various types of systems for class 1 streets, based on data available at the time of the study, are given in Table 3.

### Class 2: Major Traffic, Major Transit

The poles in the study area are located on both sides of the road in a more or less staggered configuration. The typical pole spacing ranges from 188 to 200 ft along each side; however, some poles are as close as 80 ft and as far apart as 400 ft. The calculations are based on an average pole spacing of 196 ft, a mounting height of 29.7 ft, and an overhang of 7 ft.

Because of the variations in pole spacings, the calculated uniformity values will not be achieved in a field installation. There will be short stretches where the E and L values will be higher and more uniform while other stretches with longer spacings will have lower E and L values and higher ratios of

**Table 2. Nighttime pavement illumination and luminance values for Portland streets.**

| Street Classification | Street | Illumination (footcandles) Horizontal Average | Horizontal Ave/min | Horizontal Max/min | Vertical Average | Vertical Ave/min | Vertical Max/min | Luminance (footlamberts) Average | Luminance Ave/min | Luminance Max/min |
|---|---|---|---|---|---|---|---|---|---|---|
| Regional trafficway | Northeast Columbia Boulevard | 1.1 | 3.67 | 10.67 | -[a] | - | - | 0.30 | 3.24 | 8.53 |
| Major traffic, major transit | Northeast Sandy Boulevard | 0.79 | 4.79 | 12.41 | 0.268[a]<br>0.468[b] | 3.83[a]<br>2.75[b] | 9.57[a]<br>6.94[b] | 0.21<br>0.17[c]<br>0.39[d] | 2.6 | 6.78 |
| Neighborhood collector | | | | | | | | | | |
| Major transit | North Vancouver Avenue | 0.84 | 3.52 | 7.62 | -[e] | - | - | 0.24 | 4.6 | 13.8 |
| Minor transit | Southeast Lincoln Street | 0.86 | 3.05 | 5.48 | 0.430[a]<br>0.366[b] | 7.4[a]<br>7.3[b] | 15.9[a]<br>20.0[b] | 0.18 | 7.58 | |
| Local service | Northeast 53rd Avenue | 0.36 | 33.2 | 75.0 | -[e] | - | - | 0.04 | 20.9 | 69.0 |

[a]Value for eastbound traffic lanes.
[b]Value for westbound traffic lanes.
[c]Average of six points with dry pavement.
[d]Average of same six points as in footnote c with wet pavement.
[e]Vertical illumination not measured.

**Table 3. Owning and operating costs for various lighting systems on class 1, 2, and 3 streets in Portland.**

| System | Annual Owning Cost ($/mile) | Annual Operating Cost ($/mile) | Total Annual Owning + Operating Cost ($/mile) | Annual Saving Over Present System Amount ($/mile) | Annual Saving Over Present System Percent |
|---|---|---|---|---|---|
| Class 1 | | | | | |
| Present 1000-W Hg | 0 | 4850 | 4850 | - | - |
| New 400-W HPS | 535 | 2120 | 2655 | 2195 | 45 |
| New 310-W HPS with electronic ballast | 613 | 1907 | 2520 | 2330 | 48 |
| Class 2 | | | | | |
| Present 400-W Hg | 0 | 1564 | 1564 | - | - |
| New 150-W HPS | 416 | 874 | 1290 | 274 | 18 |
| New 135-W LPS | 559 | 942 | 1501 | 63 | 4 |
| Class 3 | | | | | |
| Present 250-W Hg | 0 | 2121 | 2121 | - | - |
| New 150-W HPS | 493 | 1292 | 1785 | 336 | 16 |
| New 90-W LPS | 666 | 977 | 1643 | 478 | 23 |

Note: The lighting systems do not produce equal lighting conditions on the roadway in terms of E, L, or glare. All are greater than or equal to minimum specifications (see Table 1).

average/minimum and maximum/minimum. At the sections with very long spacings (on the order of 300 ft between poles), the values will drop to unacceptable levels. Additional poles and fixtures will therefore be required at such locations.

For class 2 streets, considerations other than lighting performance may be significant. For instance, the servicing of lights would be simpler and safer if relatively short mast arms (4-6 ft) were used so that the service truck could park at the side of the road. In addition, since these are major transit streets with many commercial businesses and passenger loading zones along the route, spill light to the side is desirable. Hence, a type III luminaire may be a good choice rather than a type II.

The color of the light is relatively unimportant insofar as traffic operations are concerned. The recommended light levels are high enough and the uniformity ratios are adequate for any available color of light (e.g., Hg, MH, HPS, or LPS) to reveal typical street objects and pedestrians to a normal attentive driver. However, color may be significant for other reasons, such as aesthetics or for color discrimination tasks.

The present luminaires and mast arms should be replaced to bring the roadway on class 2 streets up to recommended design standards. The present illumination and luminance values are not too far from the recommended average values (Table 2), but the uniformity and glare values are not acceptable. The existing poles can be used provided their remaining service life is determined to be adequate and provided that a considerable amount of variation in E and L values can be accepted due to the variations in pole spacings. The use of additional poles is recommended to improve the uniformity.

The present 400-W Hg luminaires could be replaced by 200-W or 150-W luminaires that use HPS or 135-W LPS lamps to achieve substantial energy savings and improved quality and quantity of light.

The results of cost calculations in the study project for class 2 streets are given in Table 3.

### Class 3 and Class 4 Streets

There are many variations in the conditions that affect the lighting on class 3 and 4 streets. Some areas have many trees that are not trimmed. Some areas have parking and/or sidewalks on one side only. Some areas have commercial businesses whereas others are mostly residential. In most cases, the present lighting in the study areas meets the minimum recommended quantity standards proposed in this paper. However, the quality as measured by uniformity and glare is not good.

The present poles, mast arms, wiring, and switching controls can be used as is and used in a conversion program with either (a) luminaires with retrofitted lamps and ballasts or (b) new luminaires with self-contained ballasts. A conversion will save a substantial amount of energy since the present 400- or 250-W Hg lamps can be changed to 150-, 100-, or 90-W HPS or equivalent LPS lamps, depending on the location. The present mast arms can be used provided that the struts are modified to take the occasional high wind loads. Intersection lighting along the streets should be reviewed and, where necessary, additional poles and luminaires should be installed.

The potential savings of a 150-W HPS system over a present 400- or 250-W Hg system could be quite large. One does not have to perform the detailed cost analysis to estimate the energy savings, which would be approximately in the ratio of 150/400 or 150/250.

Cost data for a class 3 street in one of the study areas are given in Table 3.

### Class 5: Local Service

A large portion of the street mileage in most cities is of the class 5 type. These are the residential and local collector streets. The lighting requirements are minimal, but they are important.

Class 5 streets in the study area are currently lighted by 175-W Hg lamps in various types of luminaires, located principally near the intersections of the local streets. Many of the block grids are 200x400 ft and have one light at midblock on the 400-ft leg and one light at the intersection.

The total amount of light on these local-service roadways from the 175-W Hg lamps is adequate, but the distribution of the light leaves much to be desired. The light is concentrated in the general vicinity of the pole, and the glare factors are quite high. A better distribution would spread the light along each roadway with a sharper cutoff at high angles. A four-way distribution at the intersections would be better, but it may not be possible to install such a system because of physical and cost limitations.

With luminaires installed on one side of the street, minimum lighting values can be met (marginally) by using the present 175-W Hg lamps or by using 70-W HPS lamps either in new commercial luminaires or retrofitted into present luminaires. It may also be possible to meet the requirements by using commercially available 55-W LPS luminaires (not evaluated).

With luminaires installed on two sides of the street in a staggered pattern, the minimum values can easily be met by using 70-W HPS lamps in commercial luminaires, 70-W HPS lamps retrofitted into existing luminaires, or 35-W LPS lamps retrofitted into existing luminaires.

Cost analysis for class 5 streets cannot be generalized because the local conditions vary greatly. For very low energy rates--such as 2.67¢/kW•h, the rate in Portland at the time of these studies--it would not be cost effective to change the 175-W Hg luminaires to new or retrofitted HPS or LPS sources because the capital and maintenance costs would be too high. However, if the improved visual conditions are considered to be necessary, then conversion of the class 5 streets is justified. As energy rates increase, the conversion becomes more viable.

### Cutout Switching Systems

The lighting systems in the study area (19) are operated on single-phase circuits and on various voltages (e.g., 120, 240, or 480 V). Groups of four to six luminaires are controlled by a photocell actuator to turn the lights on and off at preset ambient illumination levels.

Studies indicate that it would be technically feasible to operate the lamps on circuits so that every other lamp along a run could be turned on or off on a preset schedule (e.g., off at midnight and on at 5:00 a.m.). The present photocell controllers could still be used to control the basic on-off cycle based on ambient light, or a single photocell could be used at a control center.

The payback period for a switching system would be relatively short, depending on the cost of energy. For example, by changing to 150-W HPS lamps on one class 2 street, the annual savings per mile were calculated to be about \$1290. If a switching circuit were to be installed, an additional \$1290 x 0.23 = \$297/mile/year could be saved. On a 20-year, 7 percent basis, this $297 would represent an initial investment of $3143/mile. Thus, it may be very cost effective to install the system. At a higher interest rate, the plan would be even more attractive.

CONCLUSIONS

1. Nighttime roadway visibility can be greatly improved over present conditions, and energy can be simultaneously reduced.
2. The changes required in city street lighting systems to achieve improved visual conditions with substantial energy savings are cost effective.
3. Study areas show energy savings of 50-60 percent and that adequate visibility is maintained at night on city streets.
4. The overall owning and operating cost for a relighted city street will probably show a substantial reduction in cost over a present system that approximately meets existing lighting recommendations.
5. Whereas energy savings are easy to develop on a factual basis, total owning and operating costs are very difficult to develop. Each specific job must be analyzed separately by using local costs for labor, materials, interest, energy, inflation, taxes, etc., to arrive at a specific answer.

REFERENCES

1. Recommendations for the Lighting of Roads for Motorized Traffic. International Commission on Illumination, Paris, Publ. 12/2, 1975.
2. Recommendations for Roadway Lighting. Express Highway Foundation of Japan, Minato-ku, Tokyo, 1976.
3. Design Manual for Highway Illumination. Ontario Ministry of Transportation and Communications, Downsview, 1978.
4. Calculation and Measurement of Luminance and Illuminance in Road Lighting. International Commission on Illumination, Paris, Publ. 30 (TC 4.6), 1976.
5. An Analytical Model for Describing the Influence of Lighting Parameters upon Visual Performance. International Commission on Illumination, Paris, Publ. 19/2, 1981.
6. A.J. Fisher. A Review of Street Lighting in Relation to Road Safety. Australian Department of Transport, Canberra, Rept. NR/18, 1973.
7. L.L. Holladay. Fundamentals of Glare and Visibility. Journal of Optical Society of America, Vol. 12, 1926.
8. Glare and Uniformity in Road Lighting. International Commission on Illumination, Paris, Publ. 31 (TC 4.6), 1975.
9. K. Narisada. Influence of Non-Uniformity in Road Surface Luminance of Public Lighting Installations upon Perception of Objects on the Road Surface by Car Drivers. International Commission on Illumination, Paris, 17th Session, 1971.
10. B. Knudsen. Comparison of Street Lighting Codes. Light and Lighting, Vol. 57, No. 8, 1964, p. 242.
11. Warrants for Highway Lighting. Texas Transportation Institute, Texas A&M Univ., College Station, Project RF-708, 1972.
12. J. Tanner. Reduction of Accidents by Improved Street Lighting. Light and Lighting, Vol. 51, No. 11, 1958, p. 353.
13. J.M. Waldram. Street Lighting. Edward Arnold Co., London, 1952, pp. 30-32.
14. P.R. Cornwell. Appraisals of Traffic Route Lighting Installations. Lighting Research and Technology, Vol. 5, No. 1, 1973.
15. D.M. Finch and D.R. Dunlop. Forward Lighting Studies. Univ. of California, Berkeley, Final Rept. HP-54, 1971.
16. American Standard Practice for Roadway Lighting. Illuminating Engineering, Feb. 1964.
17. A.J. Fisher. Visibility of Objects Against Dark Backgrounds with Street and Vehicle Lighting. Proc., Australian Road Research Board, Vol. 4, No. 1, 1968.
18. R. Blackwell. Contrast Thresholds of the Human Eye. Journal of Optical Society of America, Vol. 36, No. 11, 1946.
19. Street Lighting Analysis: Standards and Specification for the City of Portland, Oregon. City of Portland, Vols. 1-4, Dec. 1979.
20. J. Tien. Study of City Street Lighting and Crime. U.S. Law Enforcement Assistance Administration, 1979.
21. D.M. Finch. Atmospheric Light Pollution. Proc., International Commission on Illumination, Tokyo, 1979.
22. C.L. Cottrell. Measurement of Visibility. Illuminating Engineering, Vol. 46, 1951, p. 95.
23. A.E. Simmons and D.M. Finch. An Instrument for the Evaluation of Night Visibility on Highways. Illuminating Engineering, Vol. 48, 1953, p. 517.
24. R. Blackwell. Development of Procedures and Instrument for Visual Task Evaluation. Illuminating Engineering, Vol. 65, 1970, p. 267.
25. M.S. Janoff and others. Effectiveness of Highway Arterial Lighting: Design Guide. FHWA, 1977.

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# Radio Control of Highway Lighting

RICHARD E. STARK

**Reduction in energy consumption and in public criticism of sporadic control of lighting and the need for flexibility in providing lighting under adverse weather conditions were the bases for installing radio control of freeway lighting in the Chicago metropolitan area. The problems leading to the recommendation of this type of installation are described, and the various systems available for lighting control, as well as the advantages and disadvantages of each, are discussed. The decision to use the existing Illinois Department of Transportation voice radio system as a signaling medium for control of some 166 lighting power centers was made after several trial installations of different methods of control, including radio and power-line carrier systems, were tested. The installed system is automatic in operation and has manual override. It provides instantaneous control over the entire system of some 20 000 luminaires, over individual control cabinets, or over whole freeways. Encompassed in the system are seven two-way transmitter-receiver units that feed**

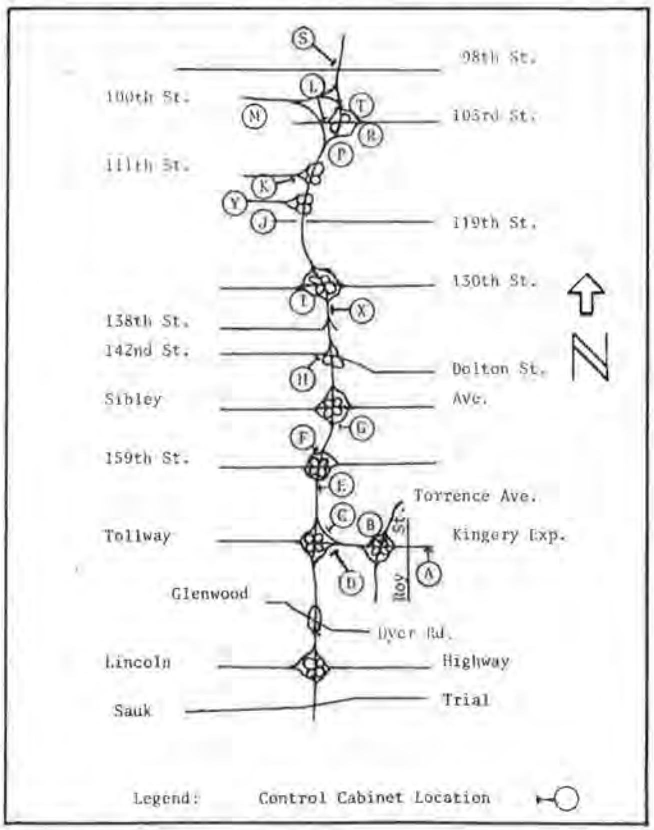
**back information about light levels and electrical conditions at these locations. Advantages of such a precise control system are the reduction of energy use; the uniform, on-time illumination of the freeways; and the ability to turn on lighting during bad weather. Ease of maintenance and reduction of driving time for maintenance workers, who can simply call for lights on, are additional benefits. A number of problems encountered in the system installation are discussed along with the potential benefits of using the basic idea as an information network for monitoring electrical parameters, traffic information, or other measurable field conditions.**

In the early 1970s, a tornado warning was issued for the Chicago area in the middle of the day. The skies were unusually dark, and a call was received from the District Highway Engineer asking that all freeway lighting be turned on. Since the 166 lighting power center distribution cabinets (referred to here as control cabinets) and some 20 000 overhead highway lights were controlled by astronomical time clocks, this request meant sending out a large force of men to turn on the lighting system. By the time all systems were energized, the sky had brightened up and it was evident that the lighting was no longer needed. The procedure then had to be reversed, a costly and ineffective solution to this visibility problem.

Since that time, a number of calls have been received from the police and other local authorities indicating a need on various freeways for illumination or optical guidance during other than normal nighttime hours. These requests were difficult to satisfy and emphasized the need to develop a more efficient method of turning on highway lighting with a minimum investment of labor.

In addition, the energy crisis provided the impetus to follow more precisely the patterns of light appreciation and depreciation (1). Originally, each control cabinet was equipped with a mechanical, astronomical time switch (astro-timer) to turn lights on or off. The astro-time switches had to be set about 15 min early for turn-on and about 15 min late for turn-off to allow for occasional dark periods due to heavy cloud cover. Photoelectric cells had been previously tried, but since the cells controlled blocks of lights, differentials between cells created an off-on pattern along the freeway. This is particularly annoying to motorists traveling at the rate of 40-55 mph, in and out of lighted areas, during the dusk and dawn periods. Much criticism was received from the motoring public and the news media concerning the lack of synchronization of lighting along the Chicago-Area Freeway System. A typical freeway with control cabinets indicated alphabetically is shown in Figure 1.

**Figure 1. Lighting control cabinets on I-94, Calumet Expressway.**




## INVESTIGATION

A number of methods of lighting control were explored, ranging from power-line carrier to solid-state astro-timer. Table 1 gives the various types of systems considered and the advantages and disadvantages of each system. The group A types of controls are local, provide no coordination, and must be constantly cleaned and adjusted to perform within expected limits. Even with the most exacting maintenance, including weekly cleaning and adjusting, the limits of operation are too variable and exclude coordination.

Group B is the cascading method (2). This is simply using one control cabinet to turn on the next by laying cable between the two and using the voltage from the first to operate a relay in the second control cabinet, closing a contactor energizing that cabinet. Each cabinet, in turn, is energized by the previous one until the whole system is energized. This requires an extensive interconnecting cable installation. As noted earlier in the system description, each control cabinet is now isolated from each other. Separate power control was the method used when the system was originally built; thus, the installation of cable to interconnect each control

**Table 1. Comparison of various types of lighting system controls.**

| | | Light Sensitive | | | | | Remote Control | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Group | Type of System | Entire System | By Area | Coordinated | Daytime Activation | Maintenance | Entire System | Individual Center | Group | Area Feedback | Installation Cost |
| A | Local photocell | Yes | No | No | No | Low | No | No | No | No | Low |
| | Local mechanical astro-time switch | No | No | No | No | Low | No | No | No | No | Lowest |
| | Local solid-state astro | No | No | No | No | Low | No | No | No | No | Still untested, not available at that time, now medium |
| B | Cascading | Yes | No | Yes | Yes | Medium | Yes | No | No | No | High cabling costs |
| C | Power-line carrier | Yes | No | Yes | Yes | Medium | Yes | Yes | Yes | No | Very high cabling costs and transmission |
| | Radio switch | Yes | Yes | Yes | Yes | Medium | Yes | Yes | Yes | Yes | Moderate to high |

cabinet would be very expensive and disruptive to traffic and roadway components.

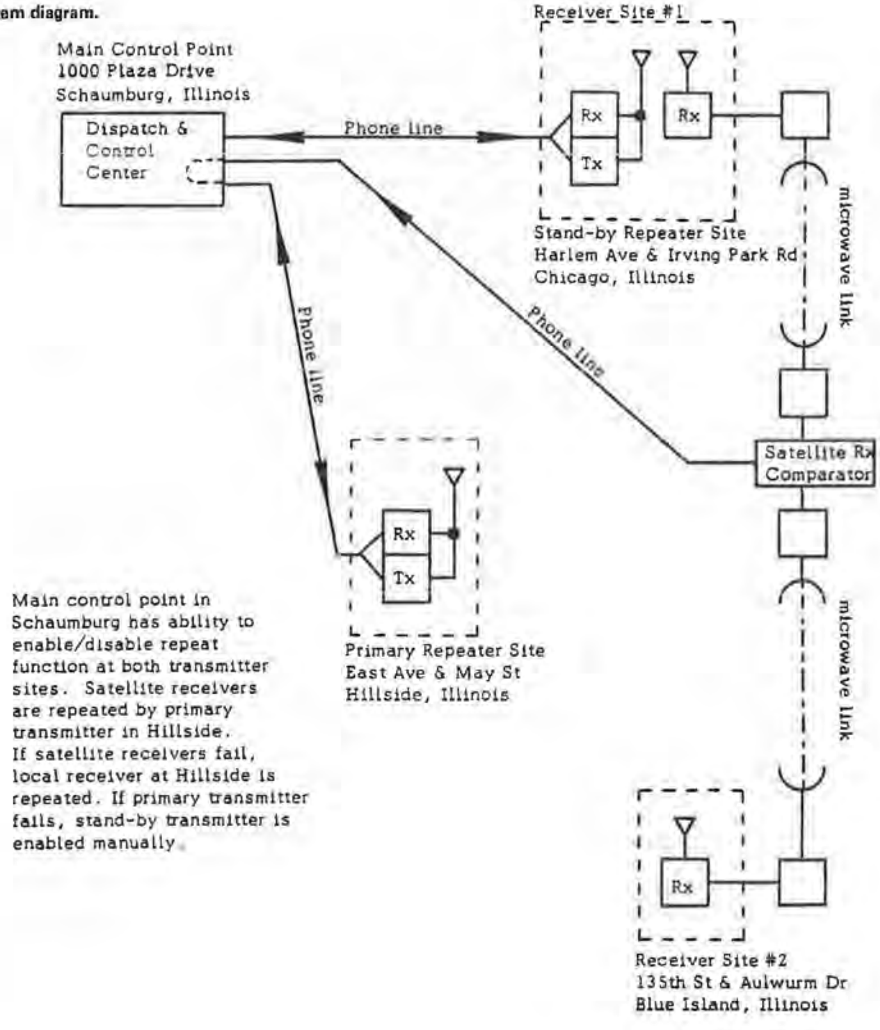
Group C systems not only are coordinated but also allow for individual control of the control cabinets by use of a coding system. The power-line carrier system is similar to the radio system in that a turn-on or turn-off signal is generated and sent to each control cabinet. The power-line carrier, however, uses the existing electrical cable system as a means of transmission rather than direct electromagnetic waves as in the radio system. This again requires the costly total interconnection of electric lines with additional isolation filters to allow signal penetration but prevent electrical power interconnection. Transmitter and receiver costs were comparable to the radio equipment, but the added interconnect cost results in a greater overall cost for this system.

## SYSTEM SELECTION

The system selected, the radio switch, had the most reasonable cost and the greatest flexibility. It was also the system that had undergone a series of tests over a period of years before the project was implemented. Tests run on early models indicated that falsing (the misinterpretation of tones by decoders) was one of the major problems. Falsing, operating on an incorrectly interpreted received signal, causes turn-ons or turn-offs at incorrect times, thereby negating proper operation. A new digital sequential coding system was developed that ensured against the probability of falsing and thus made the radio switch a possible choice for this installation.

A major factor in the final selection of a radio switch was the existing radio communication system of the Illinois Department of Transportation (District One, Highway Division). The existing radio system is composed of some 850 mobile transceivers and 10 base stations operating on 10 Federal Communications Commission (FCC) allocated frequencies. FCC rules allow for the use of voice channels for signaling if the signaling is restricted to a small percentage of the air time. Thus, the probable use of existing transmitting and receiving equipment gave the radio switch system an advantage. Eventually, channels at 151.100 MHz transmit and at 156.045 MHz receive were selected (see Figure 2). The base-station transmitter selected has the widest coverage and was reinforced by a standby transmitter in case of failure. An auxiliary generator would operate automatically under remote control in case of power failure. The backup transmitter is located at a different site (to avoid related storm and vandalism damage) and also has emergency power. Thus, the basic components of the radio switch system were already in place and, with backup features, they represented a major advantage for this type of system.

**Figure 2. System diagram.**



## SYSTEM COMPONENTS

The final system design was composed of 166 receiver switch units, 7 transmitter-receiver units, and a central control. Figure 3 is a system map that shows the various freeways involved and the location of the 7 transmitter-receiver units as well as the central control. The system serves 135 miles of freeway lighting. The individual receiver switches located at each control cabinet are designated alphabetically. One freeway (Calumet), with its 19 control cabinets, is shown in Figure 1. Each freeway has from 15 to 30 individual control cabinets. Radio switches are spread fairly uniformly over its length, and one transmitter-receiver unit installation is located in a control cabinet in the approximate middle of the freeway length. The system components and their location are shown in Figure 4.

## SYSTEM OPERATION

The basic system operation is simple. At dusk, a photocell located atop the District headquarters building in Schaumburg, Illinois, indicates by a contact closure that the ambient light level has fallen below 2 horizontal footcandles (1). This closure generates a coded signal that is automatically transmitted by radio at the first break in verbal radio traffic. The signal is received by the 166 radio switches, and this causes a relay in each control cabinet to close a contactor, which energizes the lights (see Figure 5).

The seven transmitter-receiver units located in control cabinets verify that the lighting contactor has closed at their distribution point and send a return verification signal to the headquarters central control center (see Figure 6). Each control cabinet still retains the existing astro-timer switch as a backup to the radio switch (see Figure 7). Fifteen minutes after dusk, or as set, the lights will be turned on by the astro-time clock override should the radio switch fail or the radio signal not reach the particular location. This operation also overcomes the problem of vandalism of items such as antennas.

The existing astro-time clock also serves another function in that it disables the radio switch power until 45 min before dawn the following day. This function prevents the accidental turning off of lighting by radio during the crucial nighttime hours.

Once the radio switch in each power-center distribution cabinet has been energized, a signal emanating from the headquarters control center to indicate a light level in excess of three horizontal footcandles can be received. If for some reason the radio signal fails, the astro clock will turn the lights off at some previously determined time, such as 15 min after dawn.

Since the radio switches are now energized, the lighting system may be activated at any time during

Figure 3. System map.

the day. A coding system that allows the activation of the entire system, by groups of freeways or by individual control cabinets, can be used to facilitate maintenance on the system, or the system can be activated during extremely dark periods that occur in normal daylight hours.

Additional features incorporated into the system are related to the seven transmitter-receiver units and the central control unit. Each control cabinet

Figure 4. System components.

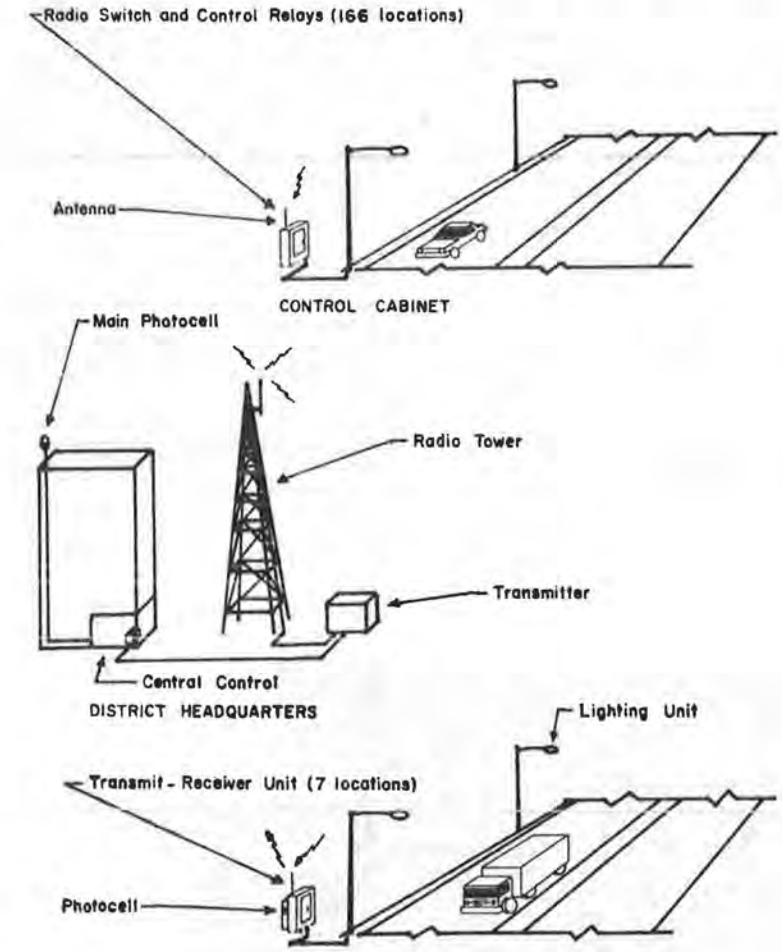



Figure 5. System block diagram.

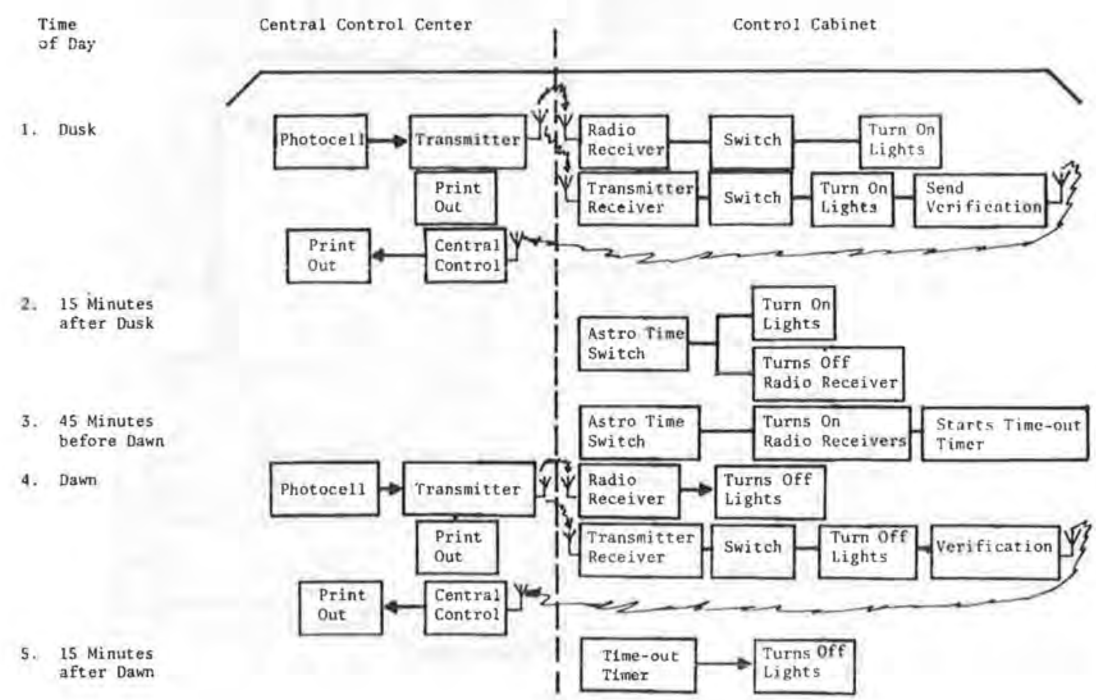


Figure 6. "Status change" verification and reporting.

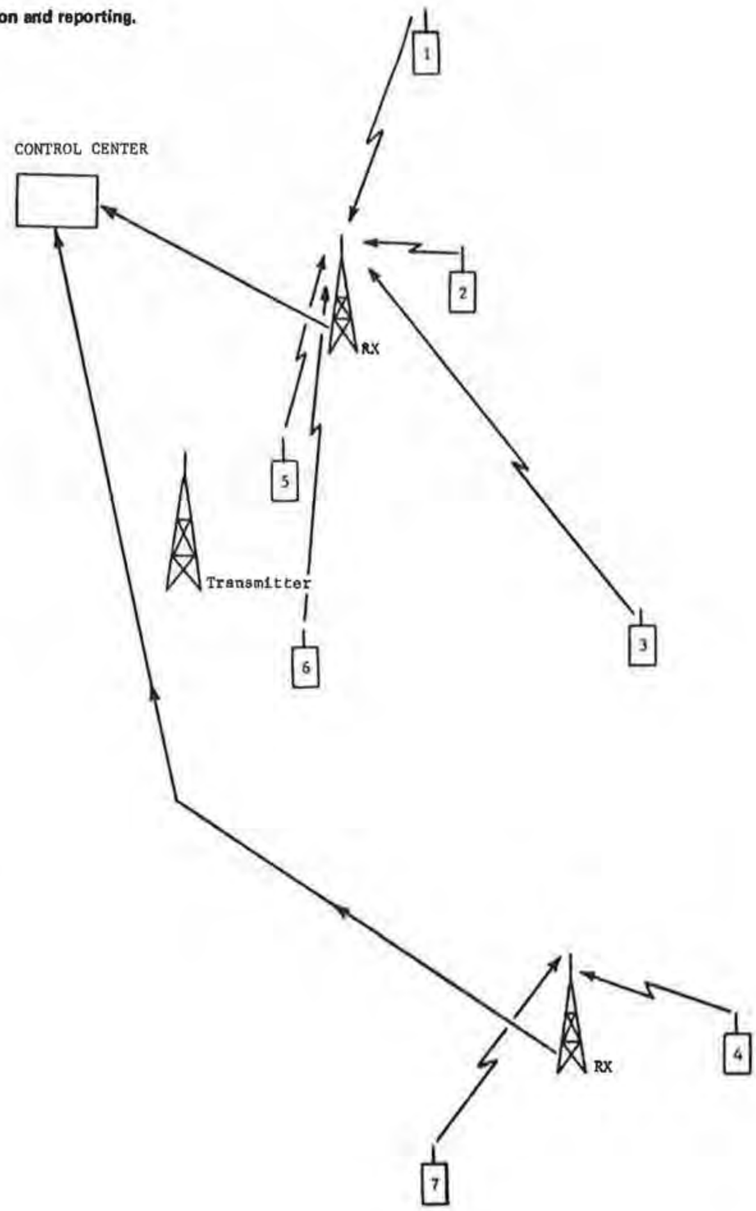
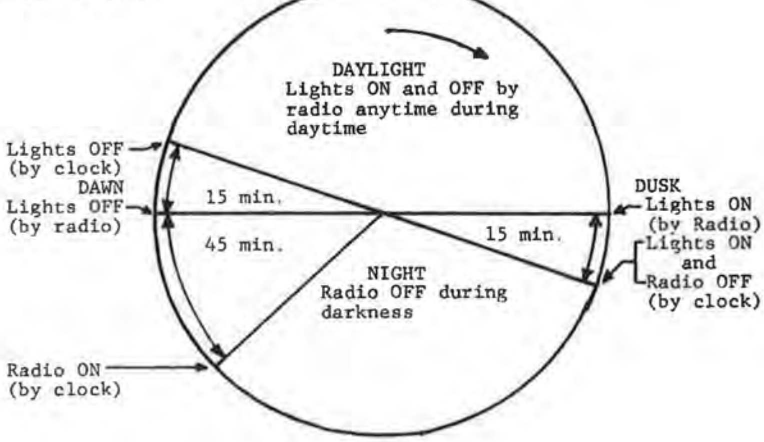



Figure 7. Twenty-four-hour operation cycle.



containing a transmitter-receiver also has a photocell that, by way of the transmitter, sends a signal to the central control in the communications room at Schaumburg when light levels fall below preset values. If these signals indicate a lower level of light on the particular freeway monitored that requires the activation of lighting, and are at sufficient variance with the central control photocell at Schaumburg (a half-hour difference between field and central control), then an alarm with digital readout will sound, alerting personnel in the Communication Center. They will then manually turn on lighting on that freeway. A special astronomical program is built into the central control unit to provide for a 1-h alarm lockout period (a half hour before and after dusk and dawn) to prevent nuisance alarms.

Each of the seven transmitter-receiver units, in addition to indicating light levels, also monitors individual circuit conditions within the control cabinet. Since the transmitter-receiver units are activated 24 hours a day, any circuit failure in these control cabinets will send an alarm to the central control located at Schaumburg headquarters. The communications dispatcher at the central control in Schaumburg then alerts the maintenance patrol person, who proceeds to the location to repair and restore the disabled circuit.

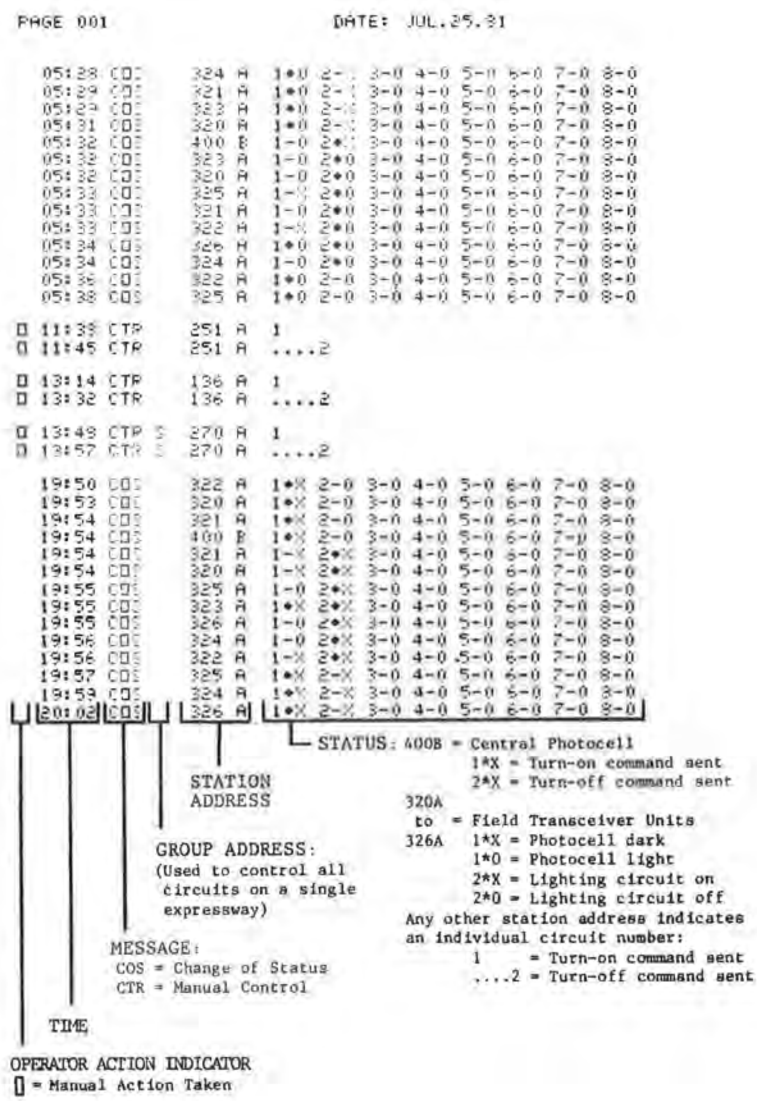
The entire system functions automatically on a daily basis unless some unusual situation occurs, such as extreme darkness during daytime hours, circuit failure at one of the transmitter-receiver units, or a request from maintenance people for turn-on at selected locations. These situations require the communication dispatcher's attention and subsequent manual operation of the system. Manual operation of the system is fairly simple and requires the dispatcher to select the proper code for individual or group turn-ons or to push one button to turn on the entire lighting system.

System operations are printed out by the central control unit on a daily basis (see Figure 8). This allows for a full check of the main photo control operation and any alarms or malfunctions of field electrical equipment at the seven transmitter-receiver locations, as well as the standard confirmations from each of the seven field units. An automatic record is also made whenever a maintenance person shuts off power to service the equipment.

## SYSTEM PROBLEMS

A number of problems developed during the installation of the system. Although it was known that the existing astro-time switches were somewhat erratic in operation, it was found that, due to inherent

Figure 8. System printout.



looseness in dial-gear meshing and relation between dial settings and actual time, there were substantial differences in timing operations. Until all clocks were brought into the shop, tested, and properly adjusted, a number of difficulties occurred in getting the radio receiver switches on early enough to enable the radio signal to perform its intended function. The preciseness of the radio operation brought attention to the impreciseness of the mechanical clock mechanism.

Since the system uses an existing voice frequency, coded signals with their accompanying sounds can be a nuisance unless they are minimized. During initial installation, problems developed before the photocells of the seven field transmitter-receiver units were properly tuned to the surrounding ambient light conditions and numerous alarms were sent to the central control. This problem was overcome by ensuring a clear viewing area for each photo control.

Another problem of growing magnitude with any type of field equipment is vandalism. Field equipment that had not been bothered for many years is now being subjected to vandalism. In the case of radio control of lighting, the only new piece of exposed equipment is the antenna. All other equipment except the seven transmitter-receiver units has been mounted within the existing locked light control boxes. The antenna, however, is symbolic of control, and it presents a unique opportunity for persons who want to disable lighting or to just vandalize, since it is so easily destroyed. We have been fortunate, however, in that most lighting control cabinets have been in place for many years and are hardly noticed in their surroundings.

## BENEFITS

Generally speaking, most of the aforementioned problems have been overcome and the system operation has provided numerous benefits. The prime reason for such an installation was the energy benefit. The system not only conserves energy but also provides savings that rapidly recover the original investment. As mentioned previously, it is no longer necessary to advance turn-on times or to retard turn-off times to compensate for overcast days. This results in a minimal 30-min saving in energy per day, and this amount is often exceeded. At the present annual kilowatt hours consumed, this amounts to more than $50 000/year. At this rate, the installation and equipment cost will be recovered in less than five years. Maintenance costs have so far been extremely low, but a figure of $6000/year was included in the estimated pay-back calculation.

A second benefit of the system is a feature termed "system equivisibility". With field sensors

**Figure 9. Operations and Communications Center report.**

**Illinois Department of Transportation**

| TIME RECEIVED | INFORMANT |
|---|---|
| 6:24PM Sun. 8/2/81 | G. Guderley |

SUBJECT:

Early turn on of Expressway lighting

LOCATION:

District 1 Expressway System

DETAILS AND NOTIFICATIONS:

Due to numerous reports from Emergency Traffic Patrol units of poor visibility resulting from heavy rains, and a request that the expressway lighting be turned on from 904-Mr. Daugherty, Emergency Traffic Patrol Foreman, the Expressway lights were turned on via the Manual "ON" switch on the Highway Lighting Controller.

VERIFIED(NAME):
POLICE & REPORT #:
E.P.V. Assist (IF ANY):

ELECTRICAL MAINTENANCE CONTRACTOR #:

| COMMUNICATIONS: | COPIES SENT TO: | INCIDENT REPORT |
|---|---|---|
| G. Guderley | Mr. Stark, Mr. Bushman | |

reporting area lighting conditions, should any unusual visibility condition develop, alarms will alert the dispatcher in the Communication Center and acceptable lighting levels can be achieved for the particular area. An example would be a heavy rainstorm in early evening that involved only one freeway. The transmitter-receiver unit for that roadway would signal low light conditions, and the dispatcher would energize that freeway lighting. Reports from field traffic personnel, executives, and others also result in turn-ons during normal daytime hours (see Figure 9).

The problem that is most noticed and causes the greatest public reaction, that of scattered turn-ons and turn-offs of highway lighting, is now replaced with instantaneous system operation. Since the system is designed to energize or de-energize whole groups of lights at one time, there are never any individual lighting units continually burning and going unnoticed for days or weeks at a time.

Finally, maintenance service time is reduced due to the ability to simply make a mobile radio call to the dispatcher for the turning on or off of any control cabinet. This saves considerable time, since these control cabinets are located off the freeway on frontage roads or cross streets. Specifically, it is necessary to energize high-mast lighting installations in order to service the luminaires. The time saved for the maintenance worker varies from as little as 10 min to 1 h round trip. As many as three other workers may be waiting during this period. Since this operation must be repeated after the repair is finished, an average of 1 h/worker is lost two to three times per week. It is estimated that an annual savings of $6000/year would be realized due to this feature.

The installation of a radio control system for highway lighting has resulted in the solution to the many aforementioned problems. The possibilities, however, for future applications of this technique are very impressive. The development of small-sized transmitters and other electronic equipment opens the way for many practical applications. One of the features of the present system, monitoring of the circuit condition, could be expanded to monitor individually each of the 20 000 luminaires. It is envisioned that a daily printout of lamp outages would be available, which would eliminate the necessity of patrolling for outages. This would result in a more efficient lamp replacement program and reduce energy and the person power required in patrolling. Installation of highway lighting systems requires substantial capital investment, which can only bring a return if the systems are properly operated and maintained. System monitoring can detect problems early and reduce the size of repairs as well as conserve energy (3).

## OTHER USES

Information of various types that can be sent to the transmitter-receiver unit can then be forwarded to the central control of the Communication Center for analysis. This might include traffic information as well as electrical parameters. Traffic information, such as detector loop output or detector information of any kind, could be sent over special frequencies. Economic comparisons of the various systems need to be made to ensure future low-cost, reliable information retrieval.

## REFERENCES


1. P.C. Box. Relationship Between Illumination and Freeway Accidents. Illuminating Engineering, May-June 1971, pp. 371-372.
2. Street Lighting Manual, 2nd ed. Edison Electric Institute, New York, 1969.
3. Roadway Lighting Handbook. FHWA, Dec. 1978.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*Abridgment*

# Programmable Roadway Lighting System as an Integral Traffic Management Component

A. KETVIRTIS

**Analysis of vehicle traffic on public roads and streets in urban and rural areas indicates that traffic density and volume vary regularly within a 24-h cycle and that there is a close relation between traffic volume and accident rate. The gap between vehicle miles of travel (demand) and available road capacity (supply) is projected to increase steadily in the future. Traffic planners are faced with the need to improve the present use of the road system without investing major sums of money. Among the objectives of such improvements is maximizing the impact of roadway lighting on traffic flow. It is recommended that roadway lighting systems include a switching flexibility that will enable traffic system operators to relate lighting levels to traffic characteristics. The operation of such variable-level lighting systems is briefly described.**

An orderly and effective traffic flow depends on a number of factors, each of which in its own way influences the totality of environmental characteristics in which the motorist is performing the driving task. In planning traffic management, therefore, it is important that these factors are clearly identified, weighed, and integrated into a unified scheme. It is obvious that, if one or more of the major contributors are ignored, success in such a system operation will be limited or, in some cases, inadequate.

The analysis of vehicle traffic on public roads and streets indicates that in urban and rural areas traffic density and volume vary regularly within a 24-h cycle. This variation establishes specific traffic-flow patterns, accentuating the morning and evening peaks as well as a slack period that normally falls between midnight and the early morning hours (Figures 1 and 2).

Based on accident distribution data within a 24-h cycle, it appears that the degree of difficulty in

Figure 1. Fatal accidents by time of day in Ontario: 1979.

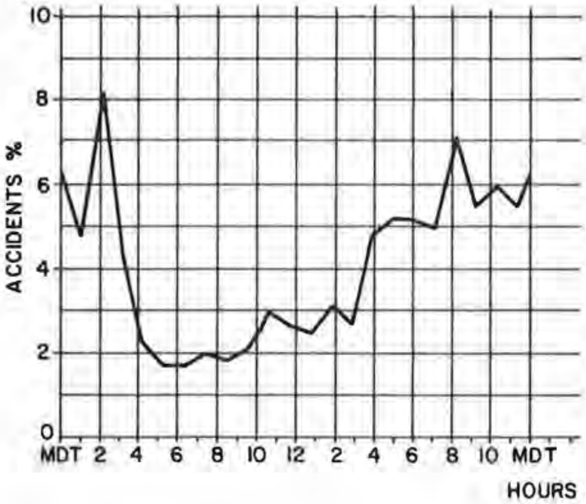
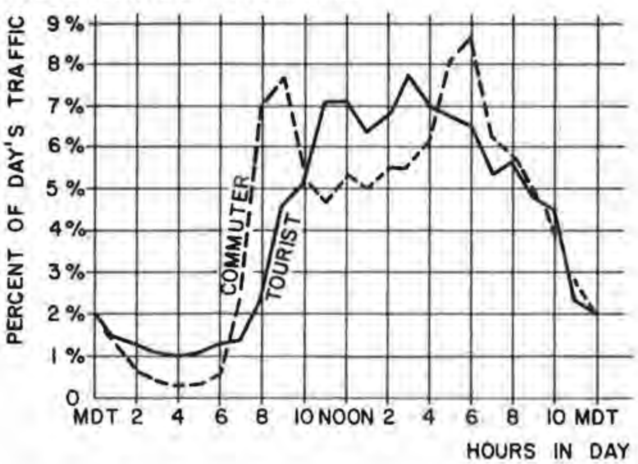



Figure 2. Hourly variations of traffic volume.




performing the driving task coincides with the peaks in traffic volume. An exception is early morning (1:00-2:00 a.m.): The high accident rate for this period perhaps can be attributed to alcohol, drugs, and fatigue.

Perhaps the close relation between volume and accident rate can be explained by the severe restriction of space for a motorist to operate a vehicle on an overcrowded road. In other words, on a crowded road the distance between vehicles is reduced and the motorist is obliged to operate his or her vehicle with considerably greater care and precision than in slack periods.

In order to maximize the impact of lighting on traffic flow, the roadway illumination should be provided with a switching flexibility to enable the traffic system operator to relate lighting levels to traffic characteristics. In other words, the variation in lighting system operation should be programmed according to changing traffic conditions by interfacing it with a traffic surveillance system and subsequently integrating it into overall traffic management.

PROJECTED GROWTH IN VEHICLE TRANSPORTATION

According to recent studies, in spite of the increasing cost of gasoline, vehicle miles of travel (VMT) in the 1980s will continue to grow at an annual rate of 3.5-4 percent. Depending on industrial and demographic activity, the annual increase in the number of registered vehicles will grow by approximately 4-5 percent.

In the State of New York, projected VMT by 1990 will increase by 37 percent (1). Vehicle registration in Ontario is increasing at the rate of 4 percent (2). Because of oil and gas developments, the Canadian provinces of Alberta, Saskatchewan, British Columbia, and possibly Newfoundland are experiencing major industrial growth, attracting migration from other provinces and overseas. The increase in vehicle traffic in these areas is also expected to reach a higher-than-average rate.

The higher cost of gasoline may suggest that, at least temporarily, VMT will decrease. However, improved engine efficiency and smaller vehicle size will encourage motorists to return to their old habits. The present 1979 model car (average) operates at 14.4-mile/gal efficiency, and by 1985 the efficiency is expected to reach 23.5 miles/gal (1). In view of the projected steady VMT growth, the consequences are clearly predictable.

Many of the present roads on this continent are already used to their capacity, and funds for construction of new facilities are diminishing. The gap between VMT (demand) and available road capacity (supply) will be widening at an increased rate. Traffic planners are faced with a new challenge: to improve the use of the present road system without investing major sums of money. The principal objective in achieving such improvements is to develop policies for implementation of more effective traffic management strategies (3).

Improvements in the use of present facilities, however, can be further increased by incorporating into the management of traffic operations other important factors that affect traffic flow. These factors are visual environment, forward visibility, conspicuity, illumination, noise control, pavement design, and vehicle and driver performance.

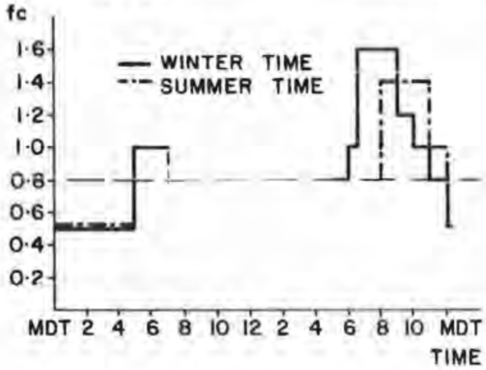
ILLUMINATION AND TRAFFIC SYSTEM OPERATION

As already stipulated, in order to maximize the impact of lighting on traffic flow, roadway illumination control equipment should include a switching flexibility that enables traffic system operators to relate lighting levels to traffic characteristics. For example, when traffic speed and volume increase and headway is reduced, visual contact with immediate objects should be much more precise; thus, the quality of illumination should be compatible with the specific driver's needs. On the other hand, in the early-morning hours traffic flow is often substantially reduced, so the driver's task becomes considerably easier; therefore, the degree of precision of required visual information is not critical.

The volume of traffic on urban expressways and freeways is normally the heaviest between 5:00 and 6:00 p.m. (Figure 2); however, since the speed at that time is slower, it is not essential in that period to increase the illumination level above the normal value. After 6:00 p.m., traffic density begins to diminish and the speed picks up. With higher speed and still relatively heavy traffic volume, the possibility of severe accidents increases; thus, visibility conditions should be improved (see Figures 2 and 3).

From the data published by numerous previous researchers (4), it appears that, by adjusting the visibility (illumination) conditions in relation to traffic volume, improvements in road capacity and motorist safety can be achieved.

Figure 3. Lighting levels related to accidents and traffic volume.




If one analyzes the traffic accident distribution within a 24-h cycle (Figure 1), it is evident that there are two peaks in the curve. The first one coincides with the early-evening rush hours, and the second occurs after midnight. The latter perhaps can be explained by driver fatigue, alcohol, and the rest cycle; therefore, it is doubtful that a higher level of illumination would have a significant influence on accident prevention at that period.

By taking into account the patterns of traffic volume and accident distribution, lighting levels can be varied as shown in Figure 3.

## OPERATION OF VARIABLE-LEVEL LIGHTING SYSTEM

As already indicated, it is desirable to design a lighting system that permits changes in the lighting level. Where traffic management systems are used, data collected by sensors monitoring traffic volume, speed, pavement conditions, and air pollution are used to control the access rate to the main traffic routes by adjusting the traffic signal cycle. This information is also used in conveying messages (variable-message signs) to drivers about difficulties ahead.

If the lighting system operation functions as a part of traffic management, some of the sensors and the electronic equipment used for traffic monitoring may be used to initiate appropriate signals for lighting system control. However, at the present time experience in operating variable-level lighting systems is very limited, and thus more research data are needed before the operating policies can be firmed up.

## PROGRAMMABLE LIGHTING SYSTEMS

Since the traffic volume on many major routes on this continent fluctuates to a ratio of 10:1, it is desirable to have a lighting system that reflects the driver's changing visibility needs. Because the traffic volume (and accidents) during rush hours represents a large portion of the total traffic count in a 24-h cycle, it is reasonable to think that the illumination level at these periods should be adjusted upward above the reference level (normal operation) (4) and that during the slack periods the level may be reduced to below the normal value.

In order to investigate the feasibility of such a lighting system, the Ontario Ministry of Transportation and Communications authorized a study (5) that resulted in a survey of electronic control equipment and ballast design that covered mainly operation of mercury vapor and high-pressure sodium lamps. From this study, it was learned that the outdoor lighting industry is in a position to provide reliable lighting system equipment for operation related to traffic needs and adaptable for interfacing with traffic surveillance equipment.

In addition to the fact that this system can be programmed to operate in relation to traffic requirements, it also provides the possibility for significant energy saving. Under normal conditions the lighting system design is based on the "maintained-level" principle. In other words, at the end of the economic lamp life, the level of illuminance provided by the system should not be less than the maintained value. For this reason, the designer is obliged to initially overdesign the system by 35-40 percent to compensate for the dirt factor and lamp-light loss factor. A system capable of controlling the lamp output can therefore be operated at the maintained level at the initial stage, which would result in approximately 20 percent average annual saving in energy consumption.

## CONCLUSIONS

If one analyzes the traffic volume within a 24-h cycle with respect to accident occurrence, it is evident that the increase in accidents in the 6:00-8:00 p.m. period is related to overcrowding of the roads. The high accident rate in the early morning hours (midnight-2:00 a.m.) can perhaps be attributed to alcohol, drugs, and fatigue.

On a crowded road, the space for operating a vehicle is often drastically reduced, which necessitates more precise visual information to guide the vehicle safely.

By relating the characteristics of the visual environment to the driving difficulty, a more effective use of visual energy (supplied by vehicle headlights and fixed-source information) can be achieved. Since the quality of the visual environment and the conspicuity within the traffic corridor are mainly controlled by fixed-source illumination, the lighting level should be programmed and its controls interfaced with overall traffic system operation.

## REFERENCES

1. D.T. Hartgen. What Will Happen to Travel in the Next 20 Years? New York Department of Transportation, Albany, Preliminary Res. Rept. 185, Aug. 1980.
2. Vehicle Accident Facts, 1978. Ontario Ministry of Transportation and Communications, Downsview, 1978.
3. A.D. May. Demand-Supply Modeling for Transportation System Management. TRB, Transportation Research Record 835, 1981, pp. 80-86.
4. A. Ketvirtis. Highway Illumination and Traffic Safety. Transport Canada, Ottawa, Ontario, TP155, May 1977.
5. Programmable Lighting Systems, Operation, and Controls. Fenco Consultants, Ltd., Toronto, Ontario, Internal Rept., 1981.

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# Driver Eye-Height Trends and Sight Distance on Vertical Curves

EUGENE I. FARBER

**A review of trends in U.S. passenger-car eye height shows only a slight decrease in eye heights since the early 1960s. Eye heights in contemporary small cars fall on or within the lower eye-height boundary for U.S. sedans. Based on the trend of the past four or five years, passenger-car eye heights do not appear to be decreasing. Analyses were performed to determine the sensitivity of stopping sight distance on vertical curves to driver eye height and other parameters entering into the stopping-sight-distance equations. Sight distance was found to be relatively insensitive to eye height. On a given hill crest, the sight distance for a driver whose eye height is 6 in lower than the design eye height (3.75 ft) is only 5 percent less than the design sight distance. On the other hand, stopping distance is very sensitive to travel speed, pavement friction, and reaction time. For example, a 1.8-mph decrease in speed reduces stopping distance by the same amount that a 6-in decrease in eye height reduces sight distance. In addition, sight distance is about 2.5 times more sensitive to obstacle height than to eye height. It is argued that reductions in travel speed since the introduction of the 55-mph speed limit compensate for any recent or projected decreases in driver eye height. In addition, because the hazard posed by a 6-in-high obstacle has not been established, it is suggested that vertical curves designed for that obstacle height probably incorporate a considerable safety factor.**

A current topic of interest in the highway safety community is the effect of the changing mix of vehicle sizes and types on the compatibility of highway and vehicle design practices. One frequently mentioned issue is the lowering of driver eye heights as a consequence of the recent and continuing trend toward smaller cars. Present highway engineering design practice to ensure adequate sight distance on hills is based on the driver eye heights that prevailed in the passenger-car fleets of the early 1960s. Several recent papers (1-4) have expressed the concern that these practices and the designs resulting from them will not provide adequate sight distance for small-car drivers.

The purpose of this paper is to analyze the role of driver eye height in determining sight distance on hill crests and, in particular, to evaluate the sensitivity of sight distance to eye height and other highway geometry and vehicle parameters. As we will see, the reductions in driver eye height that might be brought about by the advent of small cars are unimportant compared with other factors that determine sight distance and sight-distance requirements.

Much of the material presented here is based on a paper published by the Society of Automotive Engineers (SAE) (5).

## VERTICAL CURVE GEOMETRY

Hill crests are called crest vertical curves in highway engineering parlance. A crest vertical curve is the transition curve, usually a parabola, that connects the up and down grades that define the two sides of a hill. For vehicles approaching the crest of a vertical curve, the hill obstructs the view of the road ahead. Current design practices for crest vertical curves are given in the American Association of State Highway Officials (AASHO) design guide (6). Design policy for crest vertical curves is based on the need to provide drivers with adequate "stopping sight distance"--that is, enough sight distance to permit drivers to see an obstacle soon enough to stop for it under some set of reasonable worst-case conditions.

The parameters that determine sight distance on crest vertical curves are shown in Figure 1. They are change of grade (A)--that is, the algebraic difference between the slopes of the up and down grades--the horizontal length of the curve (L), and the heights above the ground of the driver's eye ($H_e$) and the obstacle to be seen ($H_O$). The length of curve required to provide a given sight distance (S) is given by the following expression:

$$L = A\,S^2/[100(\sqrt{2H_e} + \sqrt{2H_0})^2] \quad L > S \tag{1}$$

Solving for sight distance gives

$$S = 10\sqrt{L/A}\,(\sqrt{2H_e} + \sqrt{2H_0}) \tag{2}$$

For a given change of grade, the longer the curve length, the milder is the curve and the greater is the sight distance. Design values for these parameters are specified in the AASHO design guide (6).

The criterion for setting sight distance on vertical curves is the distance required to stop for an obstacle in the road. The expression used in the AASHO design guide to calculate stopping distance is

$$D = 1.467\,(RT)\,V + V^2/30f \tag{3}$$

where

- D = stopping distance (ft),
- RT = reaction time (s),
- V = speed (mph), and
- f = tire-pavement coefficient of friction.

The constants translate miles per hour into feet per second. Current practice assumes relatively poor conditions for stopping: a 2.5-s reaction time and a locked-wheel, wet-pavement stop. The effective pavement friction values assumed in the AASHO design guide range from 0.36 for stops from 30 mph to 0.29 for stops from 70 mph.

From Figure 1, it is clear that, on a given crest vertical curve, the sight distance also depends on the driver eye height and on the height of the target obstacle. On a given hill, the sight distance will increase with both target height and eye height. Conversely, the length of curve required to provide a given sight distance depends on the eye height and target height. The greater the height of either eye or target, the shorter is the length of vertical curve required to provide a given sight distance. Thus, a vertical curve design based on a given eye height will provide less sight distance to drivers with lesser eye heights. In the AASHO design guide (6), the "design" eye height for vertical curve design is 45 in and the design obstacle height is 6 in. These figures are currently under review by highway agencies.

The central issue in the design of vertical curves is the trade-off between sight distance and the cost of excavation: On a given hill, the required length of curve increases with the square of the sight distance, and the volume of soil and rock that must be excavated so that length of curve

Figure 1. Hill-crest geometry and sight distance.

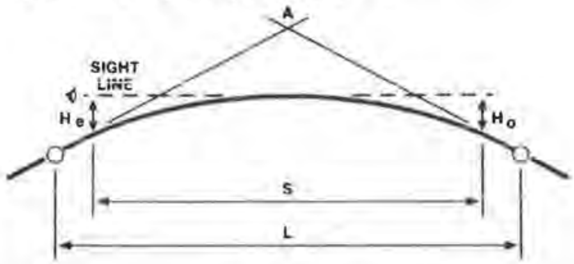



Figure 2. Length of vertical curve and associated excavation volume versus sight distance.

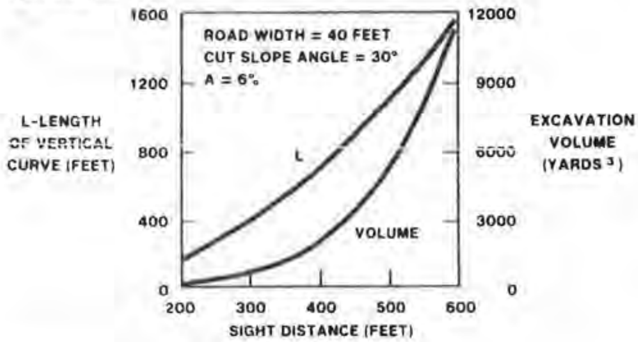
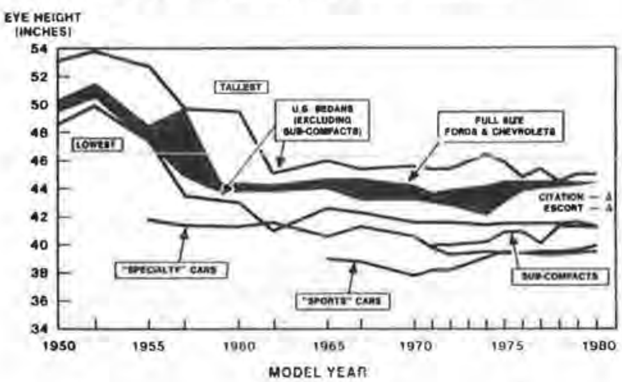



Figure 3. Eye-height trends in U.S. passenger cars: 1950-1980.




increases with the fourth power of the sight distance. These relations are illustrated in Figure 2.

## EYE-HEIGHT TRENDS

Before the analysis is discussed, it will be useful to look at actual trends in U.S. passenger-car eye heights over the past three decades. Figure 3 shows a plot of the median driver eye height (i.e., the centroid of the SAE eye ellipse) of the major domestic passenger-car lines for selected model years between 1950 and 1980. The continuous top and bottom lines show, respectively, the highest and lowest U.S. passenger-sedan eye height in each model year, excluding subcompacts. Lowest eye heights for subcompacts are shown in a separate line. The cars that constitute the domestic subcompact class are the Pinto, Bobcat, Vega, and Chevette. Also shown are the eye heights for full-sized Fords and Chevrolets and the lowest eye heights for "specialty cars" and two-seater sports cars. The specialty-car class includes vehicles such as the Thunderbird, Mustang, Camaro, and Firebird. Note that these curves show boundary values and trends for certain car classes and are not based on sales-weighted averages.

Vehicle dimensioning and measuring systems have changed since the early 1960s, and it may be that the eye heights in the first part of the graph are not compatible with later data. Nevertheless, the trend that shows the decline in eye height through the 1950s parallels the changes in roof heights over that period and is probably correct.

In any event, it is apparent that most of the decrease in passenger-car eye height over the past 30 years took place in the 1950s. By the early 1960s, minimum passenger-car eye heights were on the order of 41 or 42 in, where they remain today. Since the late 1970s, subcompact eye heights have been greater than 41 in, and the newer small cars generally have design eye heights greater than 42 in. Note that it is the specialty cars and sports cars, not the new subcompacts, that have the lowest eye heights. Based on the trend of the past four or five years, passenger-car eye heights show no signs of further decreases.

The highway design eye height specified by AASHO in 1966 was 45 in. This was regarded by AASHO as an average value. Not since the model years of the late 1950s has an eye height of 45 in been representative (i.e., toward the bottom of the range) of U.S. cars. In the absence of sales-weighted data, there is no way to determine what fraction of passenger-car eye heights is above or below a given value. However, the data in Figure 3 suggest that a representative eye height might be 41 or 42 in, if variation among drivers in seated eye heights is considered.

## SENSITIVITY TO EYE-HEIGHT CHANGES

How is visibility over the crest of a hill affected for a driver whose eye height is other than the value assumed in the design of the curve? The sensitivity of sight distance to eye height is expressed by the partial derivative, $\partial S/\partial H_e$, which gives the rate of change of sight distance with respect to eye height. The expression for this partial derivative is as follows (the derivation of this and the other partial derivatives used in the analysis is shown in Figure 4):

$$\partial S/\partial H_e = S/2\,(H_e + \sqrt{H_e H_0}) \tag{4}$$

If this expression is normalized by dividing through by sight distance, the result gives the fractional change in sight distance per unit change in eye height:

$$\partial S/\partial H_e/S = 1/2\,(H_e + \sqrt{H_e H_0}) \tag{5}$$

For the design values of eye height and object height (45 and 6 in) currently in use, the result is constant at 0.81 percent change in sight distance per inch change in eye height. Figure 5 shows a graph of Equation 4 and the change in sight distance per inch change in eye height as a function of the design sight distance. Thus, for example, on a crest vertical curve where the design sight distance is 300 ft, an inch change in eye height will produce a 2.4-ft change in the sight distance. A vertical curve designed to accommodate 60-mph traffic will provide 634 ft of sight distance (Equation 3). On such a curve, an inch change in eye height will produce a 5-ft decrease in sight distance. A convenient generalization is that a 6-in change in eye height will produce about a 5 percent change in sight distance.

This relation is illustrated in Figure 6, which

**Figure 4. Mathematical derivation of partial derivatives used in study.**

Mathematical Derivations

Partial Derivatives

Expressions are numbered consecutively. Numbers in brackets are the numbers given the expressions in the main text.

I. $\partial S/\partial H_e$

$$S = 10\,L/A\,(\sqrt{2H_e} + \sqrt{2H_o}) \qquad (1)\quad [1]$$

$$\partial S/\partial H_e = 10\sqrt{L/A}/\sqrt{2H_e} \qquad (2)$$

From (1),

$$10\sqrt{L/A} = S/(\sqrt{2H_e} + \sqrt{2H_o}) \qquad (3)$$

Substituting (3) into (2) and simplifying,

$$\partial S/\partial H_e = S/[\sqrt{2H_e}\,(\sqrt{2H_e} + \sqrt{2H_o})] \qquad (4)$$

$$\partial S/\partial H_e = S/[2(H_e + \sqrt{H_e H_o})] \qquad (5)\quad [4]$$

II. $\partial D/\partial V$

$$D = 1.467(RT)V + V^2/30f \qquad (6)\quad [3]$$

For RT = 2.5 s,

$$\partial D/\partial V = (V/15f) + 3.67 \qquad (7)\quad [6]$$

III. $\partial V/\partial H_e$

Setting S = D [(1) and (6)] gives

$$10\sqrt{L/A}(\sqrt{2H_e} + \sqrt{2H_o}) = 1.467\,(RT)\,V + V^2/30f \qquad (8)$$

and also

$$\partial S/\partial H_e = \partial D/\partial H_e$$

Recalling that

$$\partial S/\partial H_e = S/2(H_e + \sqrt{H_e H_o}) \qquad (5)$$

and differentiating implicitly with respect to $H_e$, while holding $H_o$ and RT constant,

$$S/2(H_e + \sqrt{H_e H_o}) = (1.467\,(RT) + V/15f)(\partial V/\partial H_e) \qquad (9)$$

Solving for $\partial V/\partial H_e$ and setting RT = 2.5,

$$\partial V/\partial H_e = \{S/[2(H + \sqrt{H_e H_o})]\} \times \{1/[1.467\,(RT) + V/15f]\} \qquad (10)$$

But, by assumption,

$$S = D = 1.467\,(RT)V + V^2/30f$$

Substituting D for S in (10) gives

$$\partial V/\partial H_e = \{[1.467\,(RT)V + V^2/30f] \div [1.467(RT)V + V/15f]\} \times \{1/[2(H_e + \overline{H_e H_o})]\} \qquad (11)$$

For $H_e = 3.75$, $H_o = 0.5$, RT = 2.5,

$$\partial V/\partial H_e = 0.1V[(110.1f + V)/(110.1f + 2V)] \qquad (12)\quad [7]$$

IV. $\partial D/\partial(RT)$

$$D = 1.467\,(RT)V + V^2/30f \qquad (6)\quad [3]$$

$$\partial D/\partial(RT) = 1.467V \qquad (13)\quad [8]$$

V. $\partial(RT)/\partial(H_e)$

Setting S = D gives

$$10\sqrt{L/A}\,(\sqrt{2H_e} + \sqrt{2H_o}) = 1.467(RT)V + V^2/30f \qquad (14)$$

Differentiating implicitly with respect to $H_e$ while holding $H_o$, V, and f constant gives

$$S/[2(H_e + \sqrt{H_e H_o})] = 1.467V\,[\partial(RT)/\partial H_e] \qquad (15)$$

$$\partial(RT)/\partial(H_e) = \{S/[2(H_e + \sqrt{H_e H_o}]\} \cdot (1/1.467V) \qquad (16)$$

Substituting D for S,

$$\partial(RT)/\partial(H_e) = \{[1.467(RT)V + V^2/30f] \div [2(H_e + \sqrt{H_e H_o})]\} \cdot (1/1.467V) \qquad (17)$$

Setting $H_e = 3.75$ and $H_o = 0.5$,

$$\partial(RT)/\partial(H_e) = (RT + V/44f)/10.24 \qquad (18)\quad [9]$$

VI. $\partial D/\partial f$

$$D = 1.467(RT) + V^2/30f \qquad (6)\quad [3]$$

$$\partial D/\partial f = -(V^2/30f^2) \qquad (19)\quad [10]$$

VII. $\partial f/\partial H_e$

Setting S = D,

$$10\sqrt{L/A}\,(\sqrt{2H_e} + \sqrt{2H_o}) = 1.467\,(RT)V + V^2/30f$$

Differentiating implicitly with respect to $H_e$ while holding $H_o$, V, and RT constant gives

$$S/[2(H_e + \sqrt{H_e H_o})] = -(V^2/30f)(\partial f/\partial H_e) \qquad (20)$$

Substituting D for S and solving for $\partial f/\partial H_e$,

$$\partial f/\partial H_e = -\{[1.467(RT)V + V^2/30f]/[2(H_e + \sqrt{H_e H_o})]\} \times (30f/V^2) \qquad (21)$$

Setting RT = 2.5, $H_e = 3.75$, and $H_o = 0.5$,

$$\partial f/\partial H_e = (30f/V^2)\,[(110V + V^2/f)/10.24] \qquad (22)\quad [11]$$

VIII. $\partial S/\partial H_o$

The derivation for this partial derivative is the same as for $\partial S/\partial H_e$ (4).

**Figure 5. Sensitivity of sight distance to eye height versus design sight distance.**

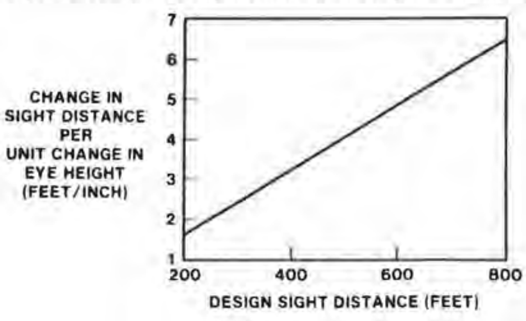
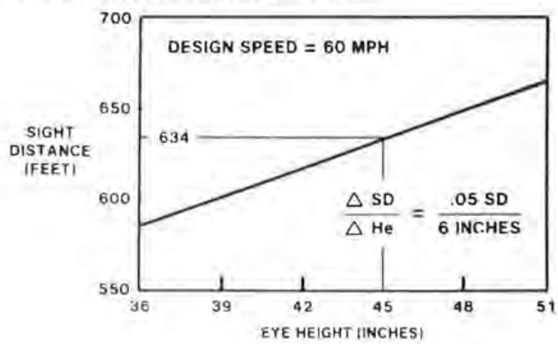



**Figure 6. Sight distance versus eye height.**




shows sight distance as a function of eye height on a 60-mph design speed vertical curve. For a driver with a 42-in eye height, the sight distance will be about 619 ft, 15 ft less than the design sight distance. With a 39-in eye height, the sight distance would be about 603 ft, 31 ft less than the design sight distance.

## SENSITIVITY TO OTHER PARAMETERS

To help put these results in perspective, the sensitivity of stopping sight distance requirements on vertical curves to the other parameters entering into Equations 2 and 3 has been calculated. These analyses are summarized in the following paragraphs. Design eye and obstacle heights of 45 and 6 in, respectively, are assumed in the calculations.

### Travel Speed

Travel speed is one of the factors entering into the expression for determining stopping distance (Equation 3), which in turn becomes the design sight distance. The partial derivative of stopping distance with respect to speed (assuming a constant f for simplicity) is given by

$$\partial D/\partial V = (V/15f) + 3.67 \tag{6}$$

This expression is plotted in Figure 7 and shows the rate of change of stopping distance per unit change in speed as a function of the travel speed. As the figure indicates, the stopping distance is quite sensitive to speed. For example, at 60 mph, each 1-mph change in speed results in a 17-ft change in stopping distance. This means that, on a vertical curve designed for 60-mph traffic, a 1-mph increase over the assumed speed will result in a sight-distance deficiency of 17 ft. From Figure 6, it can be determined that, for an eye 3.5 in lower than the design eye height, the sight distance will be 17 ft less than the design sight distance on a 60-mph vertical curve. Thus, a 1-mph increase in speed is equivalent to a 3.5-in decrease in eye height in that, in either case, the stopping distance will exceed the sight distance by 17 ft. Another way to put it is that a 1-mph decrease in speed will compensate for a 3.5-in decrease in eye height.

The sensitivity of this trade-off between speed and eye height is made explicit by setting stopping distance equal to sight distance and finding the partial derivative of speed with respect to eye height:

$$\partial V/\partial H_e = 0.1V\,[(V + 110f)/(2V + 110f)] \tag{7}$$

This parameter is plotted in Figure 8 as a function of the speed assumed for the design of the vertical curve. Equation 7 indicates how much change in eye height up or down is required to compensate for an increase or decrease in speed so as to keep the sight distance equal to the stopping distance. For example, on a curve designed for 50 mph, a 1-ft drop in eye height would decrease the sight distance as much as a 3.1-mph reduction in speed would decrease the stopping distance. On a 60-mph vertical curve, the rate is 3.6 mph per foot of eye height.

Figure 9 shows the relation between speed and eye height on a 60-mph vertical curve, given that sight distance is held equal to stopping distance. The slope of the curve is the speed/eye-height partial derivative evaluated at 60 mph, 3.6 mph/ft, or 0.30 mph/in. This graph makes it very clear that small deviations from the design speed are equivalent to large deviations from the design eye height.

Figure 7. Sensitivity of stopping distance to speed versus travel speed.

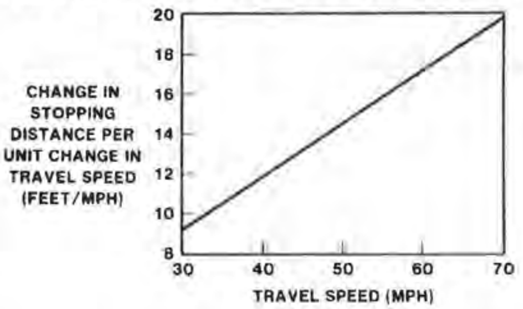



Figure 8. Design speed versus change in speed required per unit change in eye height for stopping distance to equal sight distance.

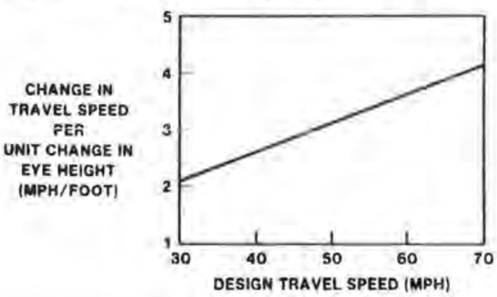



Figure 9. Travel speed versus eye height where stopping distance equals sight distance.

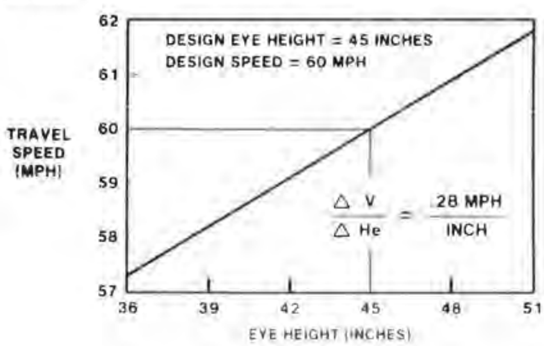
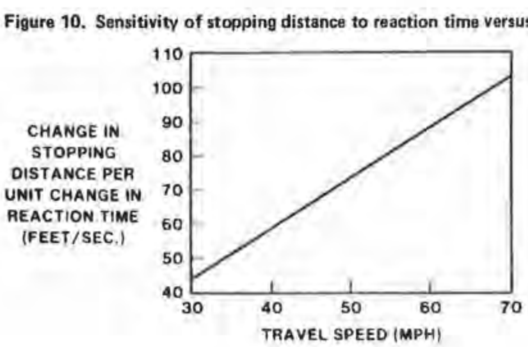



Figure 10. Sensitivity of stopping distance to reaction time versus travel speed.




## Reaction Time

Current highway engineering practice as set forth by the AASHO design guide (6) assumes a driver reaction time of 2.5 s. This is the value used to calculate stopping distance as given in Equation 3. Figure 10 shows a plot of the partial derivative of stopping distance with respect to reaction time:

$$\partial D/\partial (RT) = 1.47V \tag{8}$$

The figure shows the rate of change of stopping distance with respect to reaction time at different travel speeds. The slope is fairly steep, and at the higher speeds a small increase in reaction time has a substantial effect on stopping distance--e.g., 88 ft of stopping distance per second of reaction time at 60 mph.

Now the same procedure is followed as in the analysis of the trade-off between eye height and speed. First, it is noted that, on a given vertical curve, for any deviation from the design eye height there is a corresponding change from the design reaction time that will keep the stopping distance equal to the sight distance. For example, the sight distance on a 60-mph vertical curve for a 39-in eye height will be 31 ft less than the design sight distance (i.e., the stopping distance computed from Equation 3). The equivalent decrease in reaction time is 0.35 s because, by Equation 3, a 2.15-s reaction time results in a 603-ft stopping distance.

Figure 11 shows a plot of the partial derivative of reaction time with respect to eye height under the constraint that the stopping distance equals the sight distance. The expression for this derivative is

$$\partial RT/\partial H_e = (RT + V/44f)/10.24 \quad (9)$$

The function is almost a straight line and shows that the reaction time required to compensate for a change in eye height increases as the travel speed assumed for design purposes increases. At 60 mph, a 0.7-s decrease in reaction time would decrease stopping distance as much as a 1-ft drop in eye height would decrease sight distance. Figure 12 shows the relation between reaction time and eye height on a 60-mph crest. At this design speed, the trade-off is about 0.06 s of reaction time per inch of eye height.

## Pavement Friction

Tire-pavement friction is another parameter that enters into the stopping-distance equation (Equation 3) and thus helps determine sight-distance requirements. The sensitivity of stopping distance to pavement friction is given by the partial derivative of stopping distance with respect to the friction coefficient:

$$\partial D/\partial f = -V^2/30f^2 \quad (10)$$

This function is plotted versus design travel speed in Figure 13. As the design travel speed increases, the sensitivity of stopping distance to pavement friction also increases. At 50 mph, an increase of 0.01 in friction coefficient will produce about a 9-ft drop in stopping distance. The trade-off between eye height and pavement friction is given by setting sight distance equal to stopping distance and finding the partial derivative of pavement friction with respect to eye height:

$$\partial f/\partial H_e = -(f/V)\,[(110f + V)/10.24] \quad (11)$$

This parameter is plotted as a function of speed in Figure 14. The expression gives the change in pavement friction per unit change in eye height required to keep stopping distance equal to sight distance. The change in friction equivalent to a given eye-height change falls off rapidly with increasing speed. On a hill crest designed for 30 mph and a 45-in eye height, a 0.041 increase in pavement friction would compensate for a 6-in drop in eye height. On a 60-mph hill, the same decrease in eye height would require an increase in friction of only 0.022. The relation between eye height and pavement friction for a 60-mph vertical curve is plotted in Figure 15.

## Obstacle Height

Of all the parameters that enter into the calculations of stopping sight distance, obstacle height is the most arbitrary. The other parameter values specified in the current design guide are based on studies conducted by various highway agencies and research organizations. The 6-in obstacle height appears to have been arrived at on the basis of a trade-off between practical cost considerations and the intuitive notion that, ideally, the driver should be able to see the road surface continuously up to the stopping-distance point.

Figure 16 shows a plot of the sensitivity of sight distance to obstacle height:

$$\partial S/\partial H_0 = S/2\,(H_0 + \sqrt{H_e H_0}) \quad (12)$$

The expression is identical to Equation 4 except that $H_o$ and $H_e$ are interchanged. The eye-height/sight-distance line from Figure 5 is also plotted in Figure 16 for comparison purposes. It is obvious that sight distance is considerably more

Figure 11. Design speed versus change in reaction time per unit change in eye height required for stopping distance to equal sight distance.

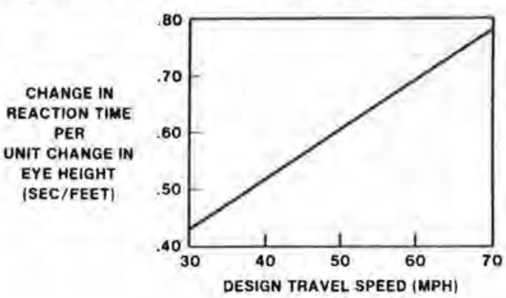



Figure 12. Reaction time versus eye height where stopping distance equals sight distance.

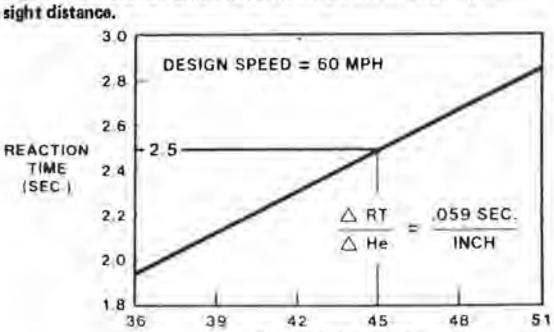



Figure 13. Sensitivity of stopping distance to coefficient of tire-pavement friction versus travel speed.

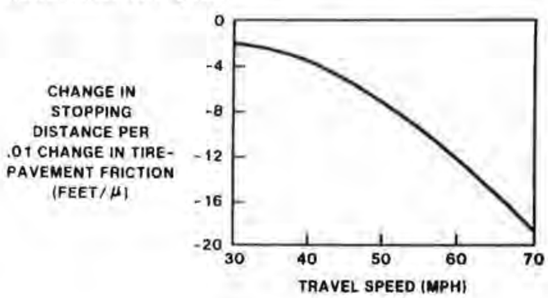



Figure 14. Change in tire-pavement friction per foot change in eye height required for stopping distance to equal sight distance.

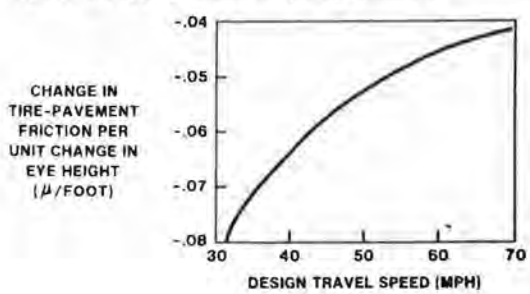


Figure 15. Tire-pavement friction versus eye height where stopping distance equals sight distance.

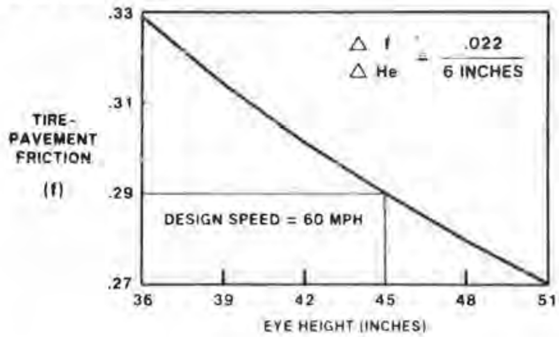



Figure 16. Sensitivity of sight distance to changes in eye height and obstacle height versus design sight distance.

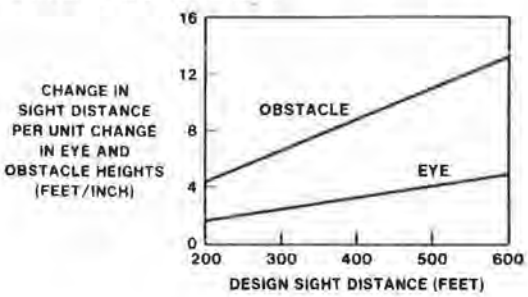
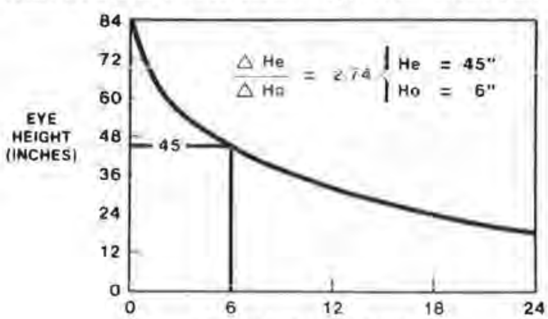



Figure 17. Eye height versus obstacle height for constant sight distance.




sensitive to obstacle height than to eye height. This is because the obstacle is so much lower than the eye. Thus, for example, on a hill crest based on current design practices, a 1-ft-high obstacle would be in view for a driver with a 32-in eye height at the same distance as a 6-in obstacle would be in view for a driver with a 45-in eye height. The relation between eye height and obstacle height for constant sight distance is plotted in Figure 17. This relation is independent of design speed. For eyeand obstacle-height values close to nominal, the rate of change is about 2.74 in of eye height per inch of obstacle height.

### Summary of Sensitivity Analysis

The results of the sensitivity analysis are summarized in the following table:

| Parameter | Reference Value | Value by Eye-Height Change: 3 Inches | 6 Inches |
|---|---|---|---|
| Speed | 60 mph | 0.9 mph | 1.8 mph |
| Friction | $0.3\mu$ | $-0.011\mu$ | $-0.023\mu$ |
| Reaction time | 2.5 s | 0.16 s | 0.32 s |
| Obstacle height | 6 in | -1.1 in | -2.41 in |

The table shows the changes in the several parameters that are equivalent to a 3- or 6-in change in eye height. In the cases of speed, reaction time, and tire-pavement friction, an "equivalent" change is the increase or decrease in the parameter required for stopping distance to equal sight distance. In the case of obstacle height, an equivalent change is the increase or decrease in obstacle height required for sight distance to remain unchanged. The sign indicates whether the change in the parameter must be in the same or opposite (-) direction as the change in eye height. The reference values are those assumed for the computation of design stopping sight distance.

## DISCUSSION OF DESIGN EYE HEIGHT

Recent Federal Highway Administration (FHWA) studies suggest that the design eye height should be lowered by from 3 to 6 in to accommodate the trend toward smaller cars (3,6,7). Our analyses have shown that sight distance on hill crests is not very sensitive to changes in eye height in this range. The effect of a 3-in drop in eye height is less than the loss of sight distance that can result from the AASHO practice (6) of rounding off calculated values of stopping sight distance to provide even numbers for design purposes--e.g., 491-475 ft.

On the other hand, the sensitivity of stopping distance to speed, reaction time, and pavement friction is so great that normal variations in these parameters simply overwhelm the effect of eye-height variation. For example, eye height would have to be more than doubled to provide adequate sight distance for a driver traveling at 65 mph on a hill crest designed for 55 mph. It thus seems likely that the decreases in travel speeds brought about by the 55-mph speed limit, and possibly also pavement friction improvements over the past decade, more than compensate for any recent or projected decreases in passenger-car eye height.

Earlier, it was observed that the AASHO design obstacle height of 6 in seems somewhat arbitrary. The authors of the AASHO design guide considered 6 in to be a reasonable minimum that would ensure the visibility of objects perhaps 1 ft in height, such as fallen trees or boulders. However, this criterion was based on intuition and engineering judgment rather than any systematic analysis of the hazard. In fact, I am not aware of any data indicating that small obstacles in the road are an important cause of accidents.

Consideration of the hazard aside, it is by no means clear that a low-criterion obstacle can ensure the visibility of small obstacles at the design sight distance. The fact that the top 6 in of an obstacle is within view at 500 ft does not mean that it can or will be seen at that distance. Six inches represents only about 3.4 min of arc at 500 ft. An object that size might not be seen for some time after it comes into view unless it contrasts strongly with the road surface. There is not much point in requiring that an obstacle be within view at a given distance if it is unlikely to be seen or noticed at that distance.

These considerations suggest the possibility that the AASHO 6-in design obstacle may be overconservative. Sight distance is much more sensitive to deviations from the design obstacle height than to deviations from the design eye height. Thus, objects larger than the design obstacle will come into view at the AASHO stopping distance for drivers whose eyes may be considerably lower than the design eye. For example, on a hill crest designed to the current AASHO practices (i.e., a 45-in-high eye and a 6-in obstacle), an 8.5-in obstacle will come into the view of a 39-in-high eye at the design sight distance; and a 15-in obstacle (e.g., the federally mandated minimum height for tail lamps) would be in view at the design sight distance for an eye only 28 in above the pavement. Accident studies or considerations of driver visual performance limitations could very well show that a 12- or 15-in-high design obstacle is more representative of real-world objects that drivers can see and need to avoid. If so, the sight distances designed to the 6-in-high target provide a considerable safety margin, and traffic safety on hill crests is not likely to be very sensitive to the changes in eye height associated with the downsizing of the passenger-car fleet.

REFERENCES

1. H.L. Anderson. Incompatibility Between Highway, Vehicles, and Drivers. Proc., American Assn. for Automotive Medicine, Oct. 1980.
2. J.J. Stapleton. Tort Liability Implications of Small Car Incompatibility with Highway Design. Proc., American Assn. for Automotive Medicine, Oct. 1980.
3. M.W. Boyd, A.C. Littleton, R.E. Boenau, and G.B. Pilkington. Determination of Motor Vehicle Eye Height for Highway Design. FHWA, Rept. FHWA-RD-78-66, April 1978.
4. G.B. Pilkington. Decision Sight Distance Requirements for the Small Car Driver. Presented at Annual Congress, Society of Automotive Engineers, Detroit, 1979.
5. E.I. Farber. Driver Eye Height and Sight Distance on Vertical Curves. Society of Automotive Engineers, Warrendale, PA, Paper 811286, 1981.
6. A Policy on the Geometric Design of Rural Highways: 1965. AASHO, Washington, DC, 1966.
7. R.L. Lee and C.P. Scott. Driver Eye Height and Vehicle Sales Trends. Office of Highway Safety, FHWA, Jan. 1981.

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# Shoulder Upgrading Alternatives to Improve Operational Characteristics of Two-Lane Highways

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**A research project was undertaken to develop upgrading warrants for use in determining when to add paved shoulders to rural two-lane roadways or when to convert two-lane roadways with paved shoulders into four-lane undivided roadways by low-cost treatments such as remarking the shoulder to indicate that it is a travel lane. The latter treatment is known as a "poor-boy" highway in Texas. The findings of one portion of the research—the effects of paved shoulders on vehicle operating characteristics—are described. Field studies were performed at 18 sites around Texas for three types of highways: (a) two-lane roadways without shoulders, (b) two-lane roadways with shoulders, and (c) four-lane, undivided roadways. Operational characteristics were recorded for more than 21 000 vehicles. Data were gathered on speed, platooning, shoulder use, and vehicle type. The findings indicate that operational benefits derived from a full-width paved shoulder increase as traffic volumes increase. These benefits are minimal at low and moderate volumes, but they become significant at volumes greater than about 200 vehicles/h. Above this volume, paved shoulders appear to increase the average speed on the roadway by at least 10 percent. They also limit the number of vehicles in platoons to less than 20 percent. No more than 5 percent of all vehicles used the shoulder at any of the sites. Conversion of the shoulder to an additional travel lane offers no apparent operational benefits until the volume reaches 150 vehicles/h. On higher-volume roads, this modification could be expected to cause average roadway speeds to increase by approximately 5 percent and limit platooning to 5 percent. Significantly, such a conversion results in more than two-thirds of the traffic using the outside (shoulder) lane.**

There are thousands of miles of existing two-lane rural roadways that are providing adequate service at low levels of vehicle flow. But, as traffic volumes grow and other characteristics change, these highways experience serious safety and operational deficiencies. It frequently becomes necessary to upgrade them to provide increased service to the higher traffic volumes.

A study was conducted in Texas to consider two improvement alternatives involving paved shoulders on rural roads: (a) adding paved shoulders to two-lane roads that previously did not have them and (b) converting two-lane highways with full-width paved shoulders into four-lane "poor-boy" roadways. The latter option is accomplished at low cost by remarking the roadway surface to indicate that the shoulder has become a travel lane. This upgrading treatment produces an undivided four-lane roadway without shoulders.

The research project was undertaken to develop upgrading warrants by quantifying the safety and operational characteristics associated with paved shoulders and by establishing the driver's understanding of the legality of driving on paved shoulders. This paper documents the findings of one portion of the research: the effects of paved shoulders on vehicle operating characteristics.

PREVIOUS RESEARCH

Many previous studies have dealt with the basic questions of shoulder design, field performance, and safety improvement; however, very few have looked at operational considerations. Several of these are reviewed below.

### Operational Studies

Several studies have attempted to identify current general practices. A national survey (1) conducted in 1973 revealed that there were four states that allowed slower traffic to drive on the shoulder in order to facilitate passing maneuvers. Unfortunately, the reasons for allowing this maneuver have been neither investigated nor reported. A comprehensive state-of-the-art review of paved shoulders was published in 1976 (2). This review noted that shoulders affected traffic in the following ways:

1. Increased lateral separation between oncoming vehicles,
2. Eased driver tension through a sense of openness and provision of space for emergency maneuvers,
3. Maintained capacity by allowing room for stopped vehicles,
4. Increased capacity where slow-moving vehicles pulled onto the shoulder to allow faster vehicles to pass (where legal), and
5. Improved traffic operations by acting as pseudo acceleration and deceleration lanes.

The results of a 1979 nationwide study on the design and use of highway shoulders (3) indicated that five states permit regular use of shoulders for slow-moving vehicles. An additional 10 states permitted such use under certain conditions.

Although these studies document general conditions when the various states allow shoulder use, they do not indicate the degree of improvement that might be expected to result from such use. Increased speed and decreased platooning are two of the benefits that probably occur, but the literature review failed to disclose any instances in which the degree of improvement had been quantified.

Messer (4) devised a technique for measuring vehicle operational characteristics during an Illinois study. While driving along a test roadway, he measured speed and location characteristics for opposing vehicle flow. Through repetitive trips along the test roadway, he acquired sufficient data to relate roadway features to operational characteristics. This field observation technique was adopted by the project staff to gather data for use in quantifying the operational improvements associated with paved shoulders in Texas.

### Driver Understanding

Concern has been expressed that motorists do not adequately understand the road-marking code and do not totally agree on the legality of driving on the shoulder. Gordon (5) examined this hypothesis in a laboratory study that involved 254 motorists from the Washington, D.C., area. He found that as many as 60 percent of the subjects thought that it was legal to use the shoulder to pass a disabled vehicle located in the main lane. Other researchers have found that, even in states where driving on the shoulder is legal, there is a great deal of driver uncertainty about which specific maneuvers are legal.

The accepted document for interpretation and application of the road-marking code is the 1978 Manual on Uniform Traffic Control Devices for Streets and Highways (MUCTD) (6), and the Uniform Vehicle Code and Model Traffic Ordinance (revised in 1968) (7) outlines accepted rules governing vehicle actions. Even though these two documents are well known to traffic engineers, the motoring public does not seem to be well informed about what shoulder markings and codes imply.

### Accident Studies

The bulk of all previous shoulder research has been safety oriented. An excellent summary was prepared by Roy Jorgensen and Associates in 1978 (8). This summary noted conflicting results from previous researchers. Several of these researchers concluded that, as the shoulder width on rural two-lane highways increases, the accident rate decreases. Others have found that under some circumstances the accident rate increases as shoulder width increases. Still others found mixed results or no relation between accident rate and shoulder width.

The apparent conflicts among previous studies can be traced mainly to small or localized data samples or failure to control all variables in the study. The majority of past studies support the concept of reduced accident rates on roadway sections with wide shoulders.

Recent research has placed emphasis on accident rate and the presence or absence of paved shoulders. Heimbach (9) performed a study on 3000 rural highways in North Carolina. He found a significantly lower accident experience and severity index for two-lane highways with 3- to 4-ft paved shoulders in comparison with similar highways without paved shoulders.

At least five research projects (8-12) have developed techniques for selecting optimum paved shoulder widths based on such factors as traffic volumes, pavement widths, traffic speeds, and construction costs. In each case, the researcher used benefits from accident reduction to justify the costs of shoulder construction.

### Summary

In general, the effects of paved shoulders on traffic operations have not been quantified. No studies could be located that predicted changes in speed, platooning, or shoulder use based on shoulder type or width. At least 15 states allow vehicle travel on the shoulder under some conditions, whereas 5 allow regular use.

Shoulder accidents have been studied in detail, and the consensus is that paved shoulders produce positive benefits by reducing accidents. Several methods have been established to choose optimum shoulder widths based on these benefits. These methods do not consider operational benefits of paved shoulders.

Although there are national documents that give explicit guidance on shoulder markings and vehicle behavior, studies have shown that motorists do not always understand or behave according to the guidance in the documents.

## SITE SELECTION

The research project was conducted on three types of highways: (a) two-lane roadways without shoulders, (b) two-lane roadways with shoulders, and (c) four-lane, undivided roadways without shoulders. Examples of each roadway type are shown in Figure 1. All roadways in Texas were screened as potential sites through use of a computer listing of the roadway geometric file, commonly referred to as the RI-2-TLOG. Control was established by defining the characteristics typical of rural Texas roadways and then rigorously screening the geometric file to locate candidate sites.

### General Study

A matrix of desired characteristics was created to stratify the sites by traffic volume, shoulder type,

Figure 1. Typical examples of three highway classes.


and number of lanes. The table below gives the 10 classifications used to allow a comparative analysis of the effects of these variables on accident rate and traffic operations. To ensure a large and representative data sample, it was desired that 10 sites in each class be studied and that each site contain 5 or more miles of consistent roadway:

| Site No. | Type of Highway | Type of Shoulder | Average Daily Traffic |
|---|---|---|---|
| 1 | Two-lane | Unpaved | 1000-3000 |
| 2 | | | 3000-5000 |
| 3 | | | 5000-7000 |
| 4 | Two-lane | Paved | 1000-3000 |
| 5 | | | 3000-5000 |
| 6 | | | 5000-7000 |
| 7 | Four-lane | Unpaved | 3000-5000 |
| 8 | | | 5000-7000 |
| 9 | | | 7000-9000 |
| 10 | | | 1000-3000 |

Once the general site criteria had been defined, the RI-2-TLOG file was carefully reviewed to obtain a list of potential rural sites that fit the requirements outlined in the table. The initial screening was a substantial undertaking that involved a manual evaluation of more than 29 000 roadway segments. A series of geometric parameters was carefully checked to ensure uniform characteristics for all eligible sites in each category.

A great deal of effort was expended in checking the eligible sites to ensure that they were typical of their respective categories. For example, average daily traffic was reviewed at each location to verify that no major changes had occurred. Pavement widths and shoulder types were scrutinized for uniformity in each class. Divided roadways were deleted from the investigation. Careful reviews were conducted in which county, highway district, and state maps were used to isolate and remove sites that contained major intersections, towns, or other factors that might bias the results of the study.

At the close of the site-screening procedure, there were 10 or more potential sites in only 6 of the categories. Class 3 had 8 sites, class 7 had 9 sites, and classes 8 and 9 had 4 sites each. The primary reasons that the desired number of sites could not be obtained were the limited mileage of roads in the categories and the rigorous screening process used to remove nonhomogeneous sites.

### Accident Study

For the accident study, 10 sites were selected where possible for each category. They were numbered, and three years of accident data were used to calculate the accident rates at each site. This study is described in detail elsewhere (13,14), and the information will not be repeated here.

### Operational Study

For the operational study, to ensure that the data sample would be representative of statewide operating conditions, the following site-selection procedure was adopted. For each of the 10 highway classes, the accident-study sites were ranked by their accident rates. The two extremes (highest and lowest rates) from each category were tentatively selected for further study. As neither extremely short nor widely separated segments lend themselves to maximizing a data-collection effort, section lengths and geometric locations were checked. Sites that did not meet these criteria were discarded and replaced by the next-ranked site in the category. Figure 2 indicates the general location in Texas of the field study sites. As shown, most of the state's geographic regions were represented in the sample. General descriptions of each site are given in Table 1.

## STUDY OF OPERATIONAL EFFECTS

The major thrust of this paper is to report on the findings of a study of operational characteristics conducted on rural Texas highways. The techniques devised by Messer (4) were used to gather several types of data in order to quantify the parameters influenced by the roadway shoulder. Traffic volume, passing opportunities, and minimization of traffic blockages were felt to be the primary factors that influence operational characteristics. For purposes of this study, shoulders were defined as being paved and 6 ft or more in width.

### Methodology

The procedure developed to collect operational data was constrained by practical considerations of mobility, accuracy, economy, and minimum distraction to motorists. Primarily, it was designed to collect five types of traffic data: traffic composition, traffic volume, vehicle speeds, lateral placement, and platooning characteristics. Roadway geometrics and other pertinent information were also recorded.

At each site, a study vehicle and two members of the research team were required to collect the field data. The vehicle was equipped with an on-board moving radar gun, a distance-measuring instrument, and several cameras. To simplify operations, most of the equipment was mounted on the dash of the car. The vehicle operator was responsible for driving the car, classifying approaching vehicles, and calling out their speeds. The responsibilities

Figure 2. Field operational study sites.

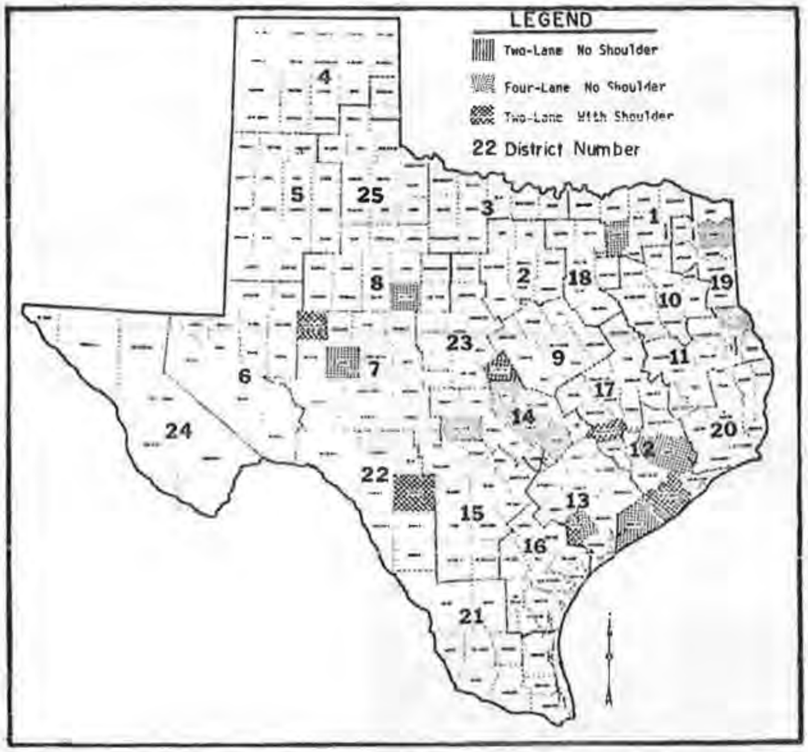



Table 1. Location and description of field study sites.

| Type of Highway | Site No. | Highway | County | Type of Curvature: Horizontal | Type of Curvature: Vertical |
|---|---|---|---|---|---|
| Two-lane, no shoulder | 101 | US-67 | Irion | Mild | Moderate |
| | 108 | US-277 | Taylor | Moderate | Moderate |
| | 205 | US-276 | Hunt | Moderate | Moderate |
| | 208 | TX-35 | Matagorda | Mild | Mild |
| | 303 | FM-2100[a] | Harris | Mild | Mild |
| | 308 | US-87 | Victoria | Mild | Mild |
| Two-lane, with shoulder | 408 | TX-105 | Washington | Moderate | Moderate |
| | 409 | TX-158 | Glasscock | Mild | Mild |
| | 501 | US-90 | Uvalde | Mild | Mild |
| | 508 | US-190 | Lampassas | Moderate | Moderate |
| | 604 | TX-35 | Brazoria | None | None |
| | 606 | US-77 | Victoria-Refugio | Mild | Mild |
| Four-lane, no shoulder | 1002 | TX-21 | Burleson-Lee | Moderate | Moderate |
| | 1009 | US-290 | Gillespie | Severe | Moderate |
| | 703 | US-290 | Bastrop | Mild | Moderate |
| | 705 | TX-29 | Burnet | Severe | Moderate |
| | 803 | US-183 | Travis | Mild | Mild |
| | 906 | US-59 | Cass | None | Moderate |

[a]Farm-to-market road.

of the study coordinator included reading longitudinal distances, recording all data, and taking photographs.

When the research team arrived at a site, features that could be easily referenced (such as intersections, bridges, and county lines) were identified to mark the ends of the study section. Several "drive-throughs" were made to familiarize the team with the site and with local traffic characteristics and to select several intermediate reference points within the section. An additional drive-through was used to videotape the site and to take 35-mm slides of the general roadway appearance. Data were collected for a 6-h period. During this time, the study vehicle was driven in a continuous circuit from one end of the section to the other. A citizens band radio was monitored to determine whether the radar had been detected or observed. Although a few speeding drivers noticed the radar (and in some instances slowed down), the overall effect on average speed was negligible. At the conclusion of the study, lane and shoulder widths were measured and recorded. Other pertinent information, such as severity of curvature and apparent aggressiveness of drivers, was also documented.

For each vehicle met by the study team, type, speed, lane position, and longitudinal placement were manually recorded. For platoons of vehicles, the speed and longitudinal position of the lead vehicle were recorded along with the number and composition of vehicles in the platoon. Vehicle classifications used in this study included passenger cars, pickups, recreational vehicles, farm vehicles, trucks, and motorcycles. These classifications were more specific than those normally associated with traffic studies; however, the research staff anticipated that agricultural or recreational vehicles might be related to shoulder-use

Figure 3. Observed shoulder use.

characteristics. Lane position referred to whether the vehicle was driving on the shoulder of the two-lane sections or in the outside lane of the four-lane sections. Longitudinal placement was used to identify locations where use of the shoulder occurred.

## Study Summary

Field data were collected at a total of 18 different sites from around the state. Three types of highways were studied: (a) two-lane highways without paved shoulders, (b) two-lane highways with paved shoulders, and (c) undivided, four-lane highways without paved shoulders. The sample included two sites from each roadway classification except for classes 8 and 9. Operational characteristics of more than 21 000 vehicles were observed and recorded. For each study site, the data were reduced, compiled, and summarized for each direction.

In the course of the field studies, many different types of shoulder use were observed. Some of these are shown in Figure 3.

## Study Results

Three major factors were analyzed by using the study data: speed, platooning characteristics, and shoulder use.

### Vehicle Speeds

Average travel speeds for all vehicles, as well as those for trucks only, are presented in Table 2 and illustrated in Figure 4. From these data, several interesting trends can be observed. Even though speeds varied between sites, they fell into a tight range of 52-62 mph. Truck speeds were examined separately to determine any restricting characteristics they might impose. Truck speeds ranged from 50 to 61 mph. For the most part, they are about the same or slightly less than the average speed on the roadway. Only at site 1009 was this not the case. The reason for this divergence is not clear; however, this particular roadway carried much less traffic than any other site even though it was a four-lane highway. The percentage of trucks at each site ranged from 5 to 15 percent. Although these numbers are higher than what might be expected for rural roadways, it should be noted that the definition of "truck" adopted for this study included single-unit as well as tractor-trailer trucks.

For two-lane roads without shoulders, the average speed drops from 61 mph at low volumes to 52 mph at high volumes (top of Figure 4). The data appear to be a reasonable approximation of a linear pattern, showing a marked decrease in speed as volume increases. The average truck speed exhibits a similar, although less pronounced, reduction. This suggests that increasing the volume on this type of highway will have less effect on truck speeds than on the speeds of other vehicles. For two-lane roads with paved shoulders, the average speed drops from 61 mph at low volumes to 57 mph at high volumes (middle of Figure 4). Again, the average truck speed exhibits a similar but less pronounced reduction. It should be pointed out that increasing volume on this type of highway will have less effect on average speeds than it will have on two-lane roads without shoulders. For the undivided, four-lane roadway (no shoulder), the average speed dropped from 59 mph at low volumes to 57 mph at high volumes (bottom of Figure 4). In this case, the reduction in average truck speed is the same. This suggests that increasing the volume on this type of highway has little effect on average vehicle speeds.

A direct comparison of the average speeds on the three types of highways is shown at the top of Figure 5. Increasing volumes have the most impact on two-lane roads without shoulders. Speeds drop rapidly as volumes increase on this type of highway. Speeds also decreased at about the same rate on two-lane roads with paved shoulders but only until the volume reached about 150 vehicles/h. Further reductions did not occur with increases in volume past this point. At volumes greater than 200 vehicles/h, the average speed on the roadways with shoulders is about 10 percent higher than it is on comparable roadways without shoulders. For the four-lane roadways without shoulders, speed did not decrease with an increase in volume. Conversion of the paved shoulder to an additional travel lane appears to increase the average speed by about 5 percent at volumes greater than 150 vehicles/h.

Average travel speeds were subjected to regression analysis to determine predictive equations. Equations 1-3 produced the best fit for two-lane roadways without shoulders, two-lane roadways with shoulders, and four-lane roadways without shoulders, respectively:

$$S = 62.2 - 0.0350\ V \tag{1}$$

$$S = 66.5 - 0.0718\ V + 0.0001\ V^2 \tag{2}$$

$$S - 59.0 - 0.0034\ V \tag{3}$$

where S is speed in miles per hour and V is volume in vehicles per hour.

The measures of effectiveness ($R^2$ and standard error) for Equation 1 are quite strong, which indicates that it is an excellent fit to the data. The equation depicts a linear relation of decreasing speed with increasing volume, which reinforces the observations for Figure 5.

Equation 2 was selected after extensive efforts to find the best fit for two-lane roadways with shoulders. The measures of effectiveness are much weaker than those for Equation 1. The difficulty was attributable to the change in the character of data at volumes greater than 150 vehicles/h. Below that point, linear regression (similar to Equation 1) fit quite nicely but, when all data were considered, linear techniques were not appropriate.

Equation 3 is linear with a very flat slope. A

**Table 2. Results of operational field studies.**

| | Direction A | | | | Direction B | | | |
|---|---|---|---|---|---|---|---|---|
| | Total Vehicles | | Trucks | | Total Vehicles | | Trucks | |
| Site No. | No. | Avg Speed (mph) | Percent | Avg Speed (mph) | No. | Avg Speed (mph) | Percent | Avg Speed (mph) |
| 101 | 217 | 62.39 | 10.14 | 58.55 | 174 | 61.71 | 13.22 | 57.96 |
| 108 | 230 | 60.70 | 19.10 | 57.80 | 219 | 61.20 | 16.00 | 59.50 |
| 208 | 472 | 57.24 | 20.34 | 56.32 | 465 | 56.72 | 22.58 | 56.32 |
| 205 | 253 | 54.20 | 10.30 | 51.80 | 257 | 53.40 | 9.30 | 54.00 |
| 308 | 637 | 54.53 | 7.22 | 54.14 | 563 | 53.30 | 5.68 | 51.75 |
| 303 | 828 | 51.80 | 13.16 | 49.90 | 738 | 53.00 | 10.03 | 57.40 |
| 409 | 382 | 62.20 | 25.00 | 60.10 | 316 | 63.40 | 20.00 | 61.60 |
| 408 | 319 | 57.99 | 14.32 | 56.59 | 413 | 60.79 | 17.19 | 57.29 |
| 501 | 470 | 58.37 | 22.13 | 57.46 | 518 | 60.56 | 23.75 | 59.74 |
| 508 | 650 | 54.73 | 12.00 | 51.05 | 629 | 54.48 | 13.51 | 55.27 |
| 604 | 1204 | 55.91 | 10.47 | 54.96 | 1039 | 56.58 | 12.32 | 54.52 |
| 606 | 1656 | 58.24 | 22.83 | 58.96 | 1465 | 56.97 | 22.80 | 56.36 |
| 1002 | 348 | 59.29 | 12.93 | 55.82 | 376 | 58.51 | 20.74 | 56.31 |
| 1009 | 107 | 60.96 | 7.48 | 62.38 | 138 | 58.70 | 13.77 | 61.00 |
| 703 | 757 | 60.14 | 15.46 | 59.41 | 674 | 61.32 | 12.61 | 59.01 |
| 705 | 674 | 52.80 | 8.90 | 52.60 | 727 | 54.60 | 10.50 | 54.50 |
| 803 | 676 | 57.01 | 12.13 | 54.92 | 643 | 58.32 | 11.20 | 56.32 |
| 906 | 1235 | 58.18 | 19.27 | 56.95 | 1143 | 59.58 | 21.61 | 58.59 |

**Figure 4. Average vehicle speeds on three types of Texas highways.**

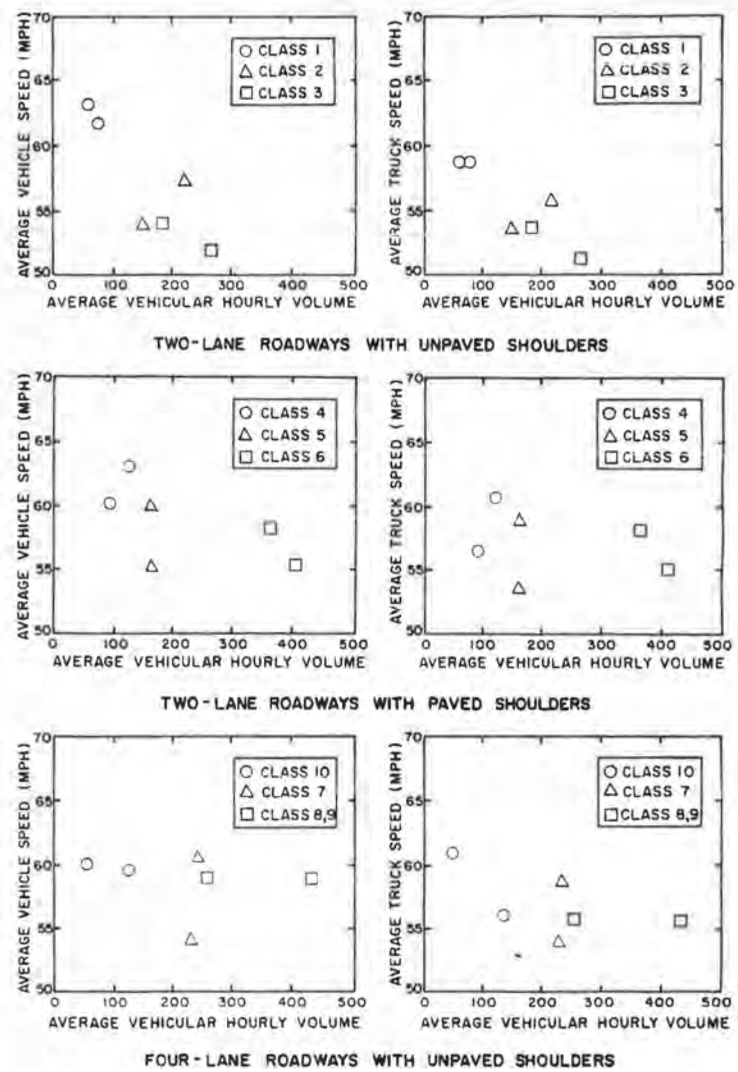
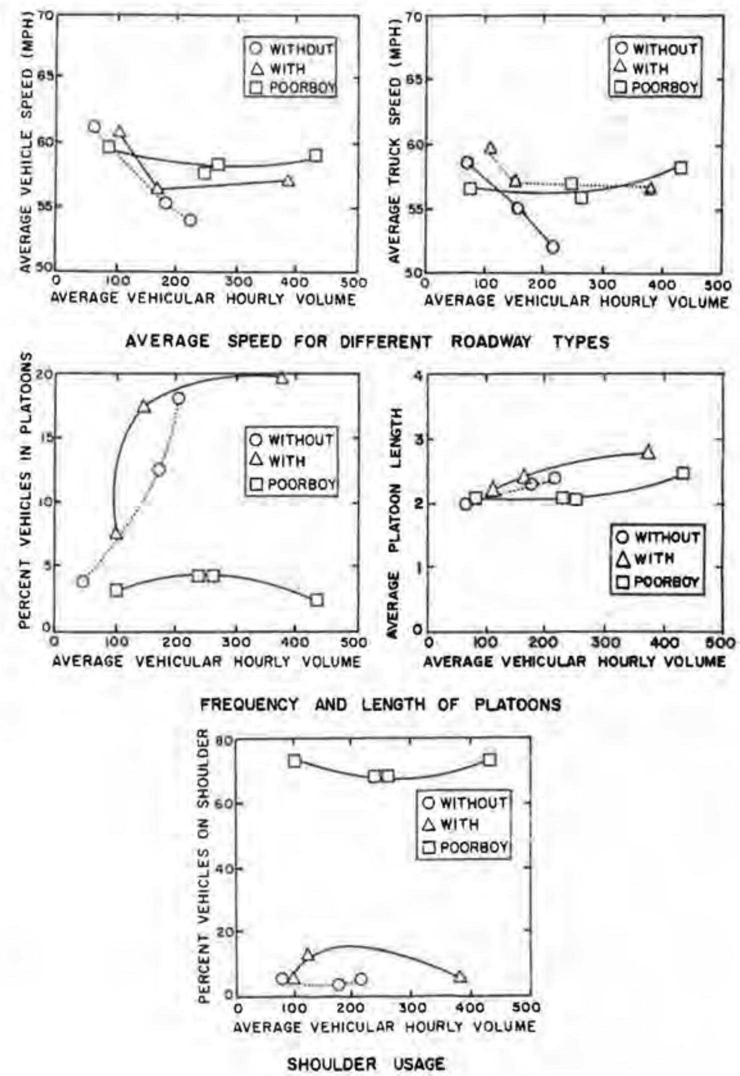


Figure 5. Operational characteristics on three types of Texas highways.




t-test (90 percent confidence level) indicated no significant difference between the designated sloped coefficient and a hypothetical slope of zero. This implies that speed on four-lane roadways without shoulders may be independent of traffic volume.

Based on these observations, the following premises have been formulated:

1. The addition of full-width paved shoulders to two-lane roadways that carry more than 200 vehicles/h will increase the average speed by at least 10 percent.
2. The conversion of a full-width paved shoulder to an additional travel lane will increase average speed by about 5 percent on roadways that carry more than 150 vehicles/h.

## Platoon Characteristics

Delay is experienced by motorists whose speeds are impeded by slower vehicles in front of them; therefore, the platooning characteristics of a roadway are an important indicator of its operational efficiency. Data from the field study were used to quantify two of these parameters: (a) average percentage of the vehicles in a platoon and (b) average length of the platoon. A direct comparison of these variables for the three types of study highways is shown in the middle of Figure 5.

As the graph on the left-hand side shows, increasing volumes have the most impact on the platoon characteristics of two-lane roadways. During conditions of low traffic flow, roadways with and without shoulders act the same. At low volumes (1000-3000 vehicles/day), the number of vehicles in a platoon ranged from 2 to 7 percent; at moderate volumes (3000-5000 vehicles/day), the range was from 12 to 17 percent; at high volumes (5000-7000 vehicles/day), more than 18 percent of the vehicles were in a platoon. At this point, the value of this parameter on two-lane roadways with shoulders began to stabilize at about 20 percent even though it was still increasing on the two-lane roadways without shoulders. This reinforces the premise that operational benefits on two-lane roadways with shoulders are not noticeable until the volume reaches 200 vehicles/h. The percentage of vehicles platooning on four-lane roadways was relatively stable for the volume levels used in the study: Typical values were 2-4 percent. These observations were confirmed by a paired t-test (90 percent confidence level).

The average length of each platoon on the three types of roadways is shown in the graph on the right-hand side of Figure 5 (middle). As expected, the average length increased with increasing volume. Comparatively, this increase was slight on the four-lane and much greater on the two-lane roadways. The data indicate parallel trends for both types of two-lane roadways and a strong degree of similarity with four-lane roadways. Surprisingly, the longest platoons occurred on the with-shoulder roadways. This observation was probably the result more of traffic volume than of roadway type. For this reason, average platoon length may not be a good measure for assessing the operational efficiency of the roadway. A more representative measure might be the percentage change in platoon length as volume increases.

### Shoulder Use

Although shoulder use on rural highways is probably greater in Texas than in any other state, its frequency of occurrence has never been determined. Therefore, one of the primary objectives of this entire study was to quantify this variable for the three roadway types. The graph at the bottom of Figure 5 illustrates the results of this effort. On the two-lane roadways without shoulders, shoulder use consisted primarily of vehicles stopped alongside the paved surface. As the lower curve shows, about 2-4 percent of the vehicles use the shoulder on this type of road. Such a low figure would be anticipated, since driving maneuvers are not normally executed on an unpaved shoulder. On the two-lane roadways with shoulders, the shoulder is used by 5-13 percent of the vehicles. This rate appears to be two or more times that of the roadways without shoulders. On the undivided, four-lane roadways without shoulders, vehicles driving in the outside lane were considered to be using the shoulder. As shown by the upper curve, between 65 and 75 percent of the vehicles use this part of the roadway. The implications of these results are discussed in the following paragraphs.

Driving patterns on two-lane roadways with shoulders and undivided four-lane roadways without shoulders are surprisingly different. Some have previously held the viewpoint that wide, paved shoulders breed sloppy driving habits and encourage use of the roadway as a pseudo four-lane roadway. The data indicate the opposite--that Texas motorists do not continually drive on the shoulder but tend to use it only in a passing situation. In fact, at a given location, only about 5 percent of the traffic uses the shoulder at all. If these same roadways are converted to four lanes, motorists will drive to the right in the outside (shoulder) lane. This modification often consists of simply restriping the roadway. If the original shoulder is not constructed to the same standards as the main lanes, the riding quality of the outside lane may be worse than the riding quality of the inside lane. Even with these conditions, drivers still retain their trained behavior of driving in the outside lane.

Thus, motorists' driving on the two types of roadways is diametrically different. Only during two maneuvers (the passing and overtaking situation or a slow-vehicle movement) do the roadways operate in the same manner. On two-lane roadways with shoulders, 95 percent of the drivers position themselves in the travel lane except when they pull onto the shoulder to let a faster vehicle through or to pass a left-turning vehicle. This leaves the paved shoulder available for a recovery area. On undivided, four-lane roadways without paved shoulders, more than two-thirds of the drivers position their vehicles in the outside (shoulder) lane and leave it only to pass vehicles that are in that lane. For all practical purposes, shoulder recovery areas no longer exist. A paired t-test confirmed that shoulder use is significantly different for poor-boy roadways than for the other study roads.

## CONCLUSIONS

Field measurements were made to quantify operational characteristics on three different rural highway types: (a) two-lane roadways without shoulders, (b) two-lane roadways with shoulders, and (c) undivided, four-lane roadways without shoulders. The results of these studies support several conclusions concerning the operational benefits attributable to the presence or absence of a paved shoulder.

As traffic volume increases, the operational benefits derived from a full-width paved shoulder increase. Although these benefits are minimal at low and moderate volumes, they are significant at volumes greater than about 200 vehicles/h. At this point, paved shoulders appear to increase the average speed on the roadway by at least 10 percent and limit the number of the vehicles that are in platoons to less than 20 percent. It appears that only about 5 percent of the total traffic actually uses the paved shoulder at any one location.

Conversion of the shoulder to an additional travel lane offers no apparent operational benefits until the volume reaches about 150 vehicles/h. On higher-volume roads, this modification could be expected to increase the average speed by about 5 percent and limit the number of vehicles that are in a platoon to less than 5 percent. Significantly, this conversion results in more than two-thirds of the traffic using the outside (shoulder) lane.

## ACKNOWLEDGMENT

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The contents of this paper reflect our views, and we are responsible for the facts and accuracy of the data presented. The contents do not necessarily reflect the official views or policies of the Federal Highway Administration. This paper does not constitute a standard, specification, or regulation.

## REFERENCES

1. N.J. Rowan. A Survey on Shoulder Design and Operation Practices. HRB, Highway Research Circular 142, April 1973, pp. 1-2.
2. J.M. Portigo. State-of-the-Art Review of Paved Shoulders. TRB, Transportation Research Record 594, 1976, pp. 57-64.
3. Design and Use of Highway Shoulders. NCHRP, Synthesis of Highway Practice 63, Aug. 1979.
4. C.J. Messer, J.M. Mounce, and R.Q. Brackett. Highway Geometric Design Consistency Related to Driver Expectancy. FHWA, Rept. FHWA-RD-79-35, 1979.
5. D.A. Gordon. Studies of the Road Marking Code. FHWA, Rept. FHWA-RD-76-59, April 1976.
6. Manual on Uniform Traffic Control Devices for Streets and Highways. FHWA, 1978.
7. Uniform Vehicle Code and Model Traffic Ordi-

nance, rev. ed. National Committee on Uniform Traffic Laws and Ordinances, Washington, DC, 1968.
8. R.E. Jorgensen and Associates. Cost and Safety Effectiveness of Highway Design Elements. NCHRP, Rept. 197, 1978.
9. C.L. Heimbach, W.W. Hunter, and G.C. Chao. Paved Highway Shoulders and Accident Experience. Transportation Engineering Journal, Proc., ASCE, Vol. 100, No. TE4, Nov. 1974, pp. 889-908.
10. P. Shannon and A. Stanley. Pavement Width Standards for Rural Two-Lane Highways. Idaho Transportation Department, Boise, Res. Project 80, Dec. 1976.
11. D.I. Davis. Shoulder Improvements on Two-Lane Roads. TRB, Transportation Research Record 737, 1979, pp. 59-60.
12. C.V. Zegeer and J.G. Mayes. Cost-Effectiveness of Lane and Shoulder Widening on Rural, Two-Lane Roads in Kentucky. Kentucky Department of Transportation, Frankfort, Res. Rept. 999, 1979.
13. D.B. Fambro, D.S. Turner, and R.O. Rogness. Operational and Safety Effects of Driving on Paved Shoulders in Texas. Texas Transportation Institute, Texas A&M Univ., College Station, Res. Rept. 265-2F, Jan. 1981.
14. D.S. Turner, D.B. Fambro, and R.O. Rogness. Effects of Paved Shoulders on Accident Rates for Rural Texas Highways. TRB, Transportation Research Record 819, 1982, pp. 30-37.

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# Before-After Accident Analysis for Two Shoulder Upgrading Alternatives

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**The Texas State Department of Highways and Public Transportation has tried several techniques to improve operating conditions on rural two-lane highways. One common treatment has been the addition of paved shoulders. An innovative treatment that provides additional capacity at a minimum cost has been the conversion of two-lane roadways with full-width paved shoulders to undivided, four-lane roadways without shoulders. Although both treatments improve traffic operations, their effect on safety has not been fully quantified. A study was conducted to establish the consequences related to safety whenever these two treatments are implemented. An accident frequency comparison for accident type by class of roadway was made for the before and after improvement time periods. Separate comparisons were made for both all accidents and nonintersection accidents. To supplement this analysis, a paired t-test was used to determine significant changes in either accident type or severity. The findings of the analysis are as follows: (a) Addition of full-width paved shoulders to a two-lane roadway is effective in reducing the total number of accidents, (b) conversion of a paved shoulder to an additional travel lane results in fewer total accidents only if traffic volume is greater than 3000 vehicles/day, and (c) the type of accident change varies with type of roadway and volume level.**

The Texas State Department of Highways and Public Transportation (TSDHPT) has tried several techniques to improve operating conditions on rural two-lane highways. The most common of these treatments has been the addition of paved shoulders. An innovative treatment that provides additional capacity at a minimum cost has been the conversion of two-lane roadways with full-width paved shoulders into undivided, four-lane roadways without shoulders. This treatment results in what is commonly known as a "poor-boy" highway and entails resurfacing and restriping, or restriping the existing pavement. Increased capacity is obtained without incurring expenses for earthwork, drainage, intersections, and structures. Although both treatments improve traffic operations, their effect on safety has not been fully quantified.

The level of safety performance is one of the major characteristics of a particular roadway. Previous safety research on shoulders and shoulder width has shown diversity in the exact relations between shoulder characteristics and accident experience. Several studies (1-3) either found mixed results or concluded that no relation existed between shoulder widths and accident rates.

Rinde (4) used a before-and-after technique to evaluate shoulder-widening projects on rural two-lane roads in California. Accident rates were reduced by 29 percent for shoulder widths of from 7 to 10 ft [average daily traffic (ADT) > 5000]. The reduction was statistically significant at the 95 percent confidence level. Head-on accidents decreased by about 50 percent, and fixed-object accidents decreased by 25 percent.

Sanderson (5) analyzed the effect that shoulder width had on the accident rates of two-lane highways in New Brunswick. He compared data on sections where the major roadway and traffic characteristics were relatively uniform. Sites were stratified and grouped into class intervals by both traffic volume and shoulder width. His results were that accident rates decreased with increasing shoulder widths.

Zegeer (6) investigated the effect of lane and shoulder widths on accident rates on rural two-lane highways in Kentucky. His stratified analysis found that wide shoulders were associated with fewer run-off-road and opposite-direction accidents.

Heimbach (7), using North Carolina data, found a significantly lower accident experience and accident severity index associated with various types of two-lane highways that had 3- to 4-ft paved shoulders when these were compared with counterpart unpaved shoulder sections.

To determine the effectiveness of the two treatments--addition of paved shoulders and conversion to four-lane poor boy--a study was undertaken to evaluate their effect on safety (8). The analysis was made for rural highways. The purpose of the analysis was to establish the safety-related consequences of adding full-width paved shoulders to a two-lane roadway or converting a two-lane highway with full-width paved shoulders to a four-lane poor-boy roadway. A previous paper (9) has described the accident effects related to the presence or absence of paved shoulders. This paper presents a comparison of accident frequency for the time periods before and after improvement.

SITE SELECTION

A stratification matrix of roadway characteristics was developed that delineated traffic volume, type of improvement, and number of lanes. Table 1 gives details on the six classifications used in the before-after analysis. To ensure a statistically valid and representative data sample, it was desired that there be a minimum of 10 sites in each classification and that each site contain 5 or more miles of geometrically consistent roadway.

The TSDHPT roadway geometric computer files, called RI-2-TLOGs, were used to screen all rural roadways in the state as potential sites. Since this process involved more than 29 000 roadway segments and 10 years of data, it was a substantial undertaking. For each segment, key geometric features in the 1977 file were checked against the same features in the 1968 file. This comparison was used to determine whether during that time period the roadway had been either reconstructed from a two-lane roadway without shoulder to a two-lane roadway with shoulder or converted from a two-lane roadway with shoulder to an undivided, four-lane roadway without shoulder. A manual examination of the two files found 390 segments (77 different sites) that had been so modified. After these roadways had been identified, their geometric files for the other eight years (1969-1976) were checked in order to determine when the modification took place. For a site to be selected, it had to have a two-year period both before and after the modification without any additional changes. In addition, candidate sites were checked for uniform cross sections, consistent traffic volumes, and standard geometric features. Roadways that did not meet these criteria were discarded.

The results of this initial screening process indicated that there was not a uniform distribution of candidate sites among the different roadway classes. There were more than enough class 1 roadways (shoulders added to a low-volume, two-lane highway) suitable for this study. The number of class 2 roadways (shoulders added to a midvolume, two-lane highway) was sufficient to choose sites that met most of the selection criteria. Class 3 roadways (shoulders added to a high-volume, two-lane highway) were virtually nonexistent since only three possibilities were found. Sites suitable for evaluating the conversion from a two-lane roadway with shoulder to an undivided, four-lane roadway without shoulder were limited by two conditions: Either the conversion took place less than three years after a shoulder had been added, or it was directly converted from a two-lane roadway without shoulder. Several additional sites were dropped from the sample because the change took place after 1975; i.e., two years of accident data after the conversion were not available. In addition, several sites were less than 5 miles in length. Because of these problems and the relatively small sample size, few additional sites were eliminated. The process by which the study sites were selected is summarized in Table 2.

In summary, an adequate number of low-volume (class 1 and 4) roadways were found that either met or exceeded the minimum selection criteria. The number of moderate-volume (class 2 and 5) roadways had been expanded to a desirable size by including several roadway segments that were less than 5 miles in length. The number of high-volume (class 3 and 6) roadways was inadequate, and both samples encompassed a broad range of conditions. An initial finding of this study is that, even with a large number of candidate roadway segments, the selection criteria restricted the number of sites eligible for study to a relatively small number. However, even with this shortcoming, the sample size was larger and more uniform than those used in previous before-after accident studies (4).

DATA COLLECTION

For each study location, the TSDHPT accident files were used to obtain the accident histories for each site during the two years before and two years after the modification took place. At this time, checks for both milepoint compatibility and construction-related accidents were made. This ensured that data were collected for the same section of roadway and also that the modification was completed within a one-year time period. During this process, several adjustments to the data set were required to account for extended construction activity. In some cases, this necessitated the use of up to six years of accident data.

Compilation of the accident data was another labor-intensive task. Each accident record had to be manually transcribed from a microfiche card reader to a coding form before it could be placed in a computer data file. Each record contained more than 30 variables. To compound this problem, minor changes in both format and variable descriptions occurred from time to time. Conversion to a compatible data base was done manually. Although each item of data was coded, only the information that was relevant to this type of study was analyzed. The variables that were analyzed are listed below:

1. Accident type--Angle, head-on, right-turn, left-turn, rear-end, same direction, fixed-object, run-off-road, animal, other, and total;
2. Severity measures--Personal injury and fatality (PI&F) accidents, fatality accidents, number of injuries, and number of fatalities;
3. Time of day--Daytime, nighttime, and total; and

**Table 1. Site classification.**

| Classification No. | Modification Condition | ADT |
|---|---|---|
| 100 | Add paved shoulders to two-lane highway | 1000-3000 |
| 200 | Add paved shoulders to two-lane highway | 3000-5000 |
| 300 | Add paved shoulders to two-lane highway | 5000-7000 |
| 400 | Convert two-lane with shoulders to four-lane without paved shoulders | 1000-3000 |
| 500 | Convert two-lane with shoulders to four-lane without paved shoulders | 3000-5000 |
| 600 | Convert two-lane with shoulders to four-lane without paved shoulders | 5000-7000 |

**Table 2. Site selection.**

| Class | Candidate RI-2-TLOG Segments | | No. of Potential Sites | No. of Eligible Sites | Sites Selected | |
|---|---|---|---|---|---|---|
| | No. | Length (miles) | | | No. | Length (miles) |
| 1 | 10 515 | 7101 | 260 | 18 | 16 | 135 |
| 2 | 3 969 | 1885 | 51 | 18 | 11 | 68 |
| 3 | 1 346 | 516 | 32 | 3 | 3 | 11 |
| 4 | 453 | 187 | 15 | 15 | 13 | 78 |
| 5 | 554 | 223 | 19 | 17 | 11 | 68 |
| 6 | 403 | 80 | 13 | 6 | 6 | 34 |
| Total | 27 240 | 9992 | 390 | 77 | 60 | 394 |

4. Location--All and nonintersection.

Finally, it should be noted that accidents not necessarily related to roadway type were excluded from the data set.

For comparative purposes, a frequency analysis was run for each site on a yearly basis--two years before and two years after modification. Accident frequencies for the after conditions were adjusted to account for changes in ADT. Separate analyses are provided for total accident frequencies and nonintersection accident frequencies.

## FREQUENCY ANALYSIS

### All Accidents

The results of the frequency analysis for the all-accident data set are summarized in Tables 3 and 4. Although nine types of accidents were studied, they have been grouped into three broad categories for this discussion: multivehicle accidents, run-off-road and fixed-object single-vehicle accidents, and other single-vehicle accidents. A more detailed breakdown is not necessary since both categorization schemes support the same conclusions. Major findings from this study are discussed below.

When full-width paved shoulders were added to a two-lane roadway (Table 3), total accidents decreased in number. This was true for all volume levels studied. In the low-volume category (1000-3000 vehicles/day), there were a total of 16 sites. The average volume on these roadways was 1450 vehicles/day in the after condition. Although the number of total accidents decreased, only single-vehicle run-off-road and fixed-object accidents contributed to this reduction. There were 11 sites in the moderate-volume category (3000-5000 vehicles/day). The average volume on these roadways was 2550 vehicles/day in the before condition and 2730 vehicles/day in the after condition. Several of the sites were less than 5 miles in length. Although the total number of accidents again decreased, both multivehicle and single-vehicle run-off-road and fixed-object accidents contributed to this reduction. There were only three sites in the high-volume category (5000-7000 vehicles/day). The average volume on these roadways ranged from 3840 vehicles/day in the before condition to 4880 vehicles/day in the after condition. All three sites were less than 5 miles in length. As before, the number of total accidents decreased. However, in this situation, multivehicle accidents were the primary contributor to this reduction.

When a two-lane roadway with paved shoulder was converted to an undivided, four-lane roadway without shoulder (Table 4), total accidents decreased only for volume levels greater than 3000 vehicles/day. The low-volume category included 12 sites. The average volume on these roadways was 2000 vehicles/day in the before condition and 2200 vehicles/day in the after condition. Although the number of single-vehicle run-off-road and fixed-object accidents decreased, there was a small increase (3 percent) in the total number of accidents. In the moderate-volume category, there were 11 sites. The average volume on these roadways changed from 2910 vehicles/day in the before condition to 3310 vehicles/day in the after condition. In this category, all three types of accidents decreased in number. There were only six sites in the high-volume category. The average volume on these roadways was 4100 vehicles/day in the before condition and 5130 vehicles/day in the after condition. As in the previous category, all three types of accidents decreased in number.

### Nonintersection Accidents

The results of the frequency analysis for the nonintersection-accident data set are summarized in Tables 5 and 6. Intersection-related accidents have been deleted. Categories for accident types and the sample of study sites are the same as those in the previous discussion. For this reason, their descriptions will not be repeated. Results from the analyses of the two data sets are remarkably similar. The following paragraphs describe the probable causes of these trends.

When full-width paved shoulders are added to a two-lane highway (Table 5), the number of total accidents can be expected to decrease in number. This was true for all volume levels studied. Both the magnitude of the reduction and the types of accidents contributing to it changed with increasing volume. The biggest percentage savings occur at low traffic volumes; however, single-vehicle accidents were the only type of accidents that decreased in number. Since they constitute almost 70 percent of the total accidents in the "before" condition, these results are not surprising. Accidents that occur at these volume levels are often the result of inattentiveness brought on by a low driver workload. Typically, these incidents involve one driver who for some reason loses control of his or her car and runs off the road. Adding paved shoulders to the roadway provides more surface area for motorists to recover from this type of mistake. This results in a decrease in the expected number of single-vehicle accidents.

At moderate volumes, accident reductions are less than half those of the lower-volume category. In this case, both single-vehicle and multivehicle accidents decreased in number. The percentage of single-vehicle accidents in the "before" condition has decreased to about 60 percent. The additional traffic has increased the driver's workload and made him or her more attentive, but at the same time it has increased the driver's probability of hitting another car. Although paved shoulders still provide recovery area, they are now being used for accident avoidance maneuvers. Thus, the expected number of both accident types decreases. At high traffic volumes, accident reductions fall midway between those for the other two volume categories. It is interesting that only multivehicle accidents contributed to this saving. The percentage of single-vehicle accidents in the "before" condition has decreased to 40 percent. High traffic volumes have caused the driver's workload to become extremely heavy. As a result, accidents caused by inattention are infrequent. Since most incidents involve more than one vehicle, any safety benefits that result from the addition of paved shoulders must come from a reduction in the number of multivehicle accidents.

When a two-lane roadway with paved shoulder is converted to an undivided, four-lane roadway without shoulder (Table 6), total accidents can be expected to decrease in number whenever the road carries more than 3000 vehicles/day. If the volume is lower than this, accident frequencies will probably increase. At these low volumes, both single-vehicle and multivehicle accidents can be expected to increase in number. As described previously, accidents that occur at these volume levels are often the result of inattentiveness brought on by a low driver workload. Conversion of the shoulder to a travel lane adds to this false feeling of security and reduces the area available for vehicle recovery. Therefore, it is not surprising that accident rates increased after modification. This type of improvement does reduce accidents at higher volume levels. This saving increases with increasing volumes. It is

**Table 3. Safety benefits of adding shoulders to two-lane roadway: all accidents.**

| Traffic Volume | Type of Accident | No. of Accidents Before | After | Change (%) |
|---|---|---|---|---|
| 1000-3000 | Multivehicle | 69 | 69.5 | +0.7 |
| | Single vehicle | | | |
| | Run-off-road and fixed-object | 74 | 34.2 | −53.8 |
| | Other | 29 | 31.7 | +9.3 |
| | Total | 172 | 135.4 | −21.3 |
| 3000-5000 | Multivehicle | 92 | 82.3 | −10.5 |
| | Single vehicle | | | |
| | Run-off-road and fixed-object | 77 | 62.7 | −18.6 |
| | Other | 31 | 39.1 | +26.1 |
| | Total | 200 | 184.1 | −8.0 |
| 5000-7000 | Multivehicle | 44 | 33.9 | −23.0 |
| | Single vehicle | | | |
| | Run-off-road and fixed-object | 17 | 14.9 | −12.4 |
| | Other | 7 | 8.2 | +17.1 |
| | Total | 68 | 57.0 | −16.0 |

**Table 4. Safety benefits of converting to four-lane poor-boy highway: all accidents.**

| Traffic Volume | Type of Accident | No. of Accidents Before | After | Change (%) |
|---|---|---|---|---|
| 1000-3000 | Multivehicle | 61 | 68.2 | +11.8 |
| | Single vehicle | | | |
| | Run-off-road and fixed-object | 90 | 75.3 | −16.3 |
| | Other | 33 | 45.3 | +37.3 |
| | Total | 184 | 188.7 | +2.6 |
| 3000-5000 | Multivehicle | 104 | 77.0 | −26.0 |
| | Single vehicle | | | |
| | Run-off-road and fixed-object | 83 | 71.6 | −13.7 |
| | Other | 42 | 35.4 | −15.7 |
| | Total | 229 | 184.0 | −19.7 |
| 5000-7000 | Multivehicle | 80 | 72.7 | −9.1 |
| | Single vehicle | | | |
| | Run-off-road and fixed-object | 57 | 35.8 | −37.2 |
| | Other | 26 | 24.4 | −6.2 |
| | Total | 163 | 132.9 | −18.5 |

**Table 5. Safety benefits of adding shoulders to two-lane roadway: nonintersection accidents.**

| Traffic Volume | Type of Accident | No. of Accidents Before | After | Change (%) |
|---|---|---|---|---|
| 1000-3000 | Multivehicle | 35 | 36.4 | +4.0 |
| | Single vehicle | | | |
| | Run-off-road and fixed-object | 58 | 26.1 | −55.0 |
| | Other | 27 | 25.1 | −7.0 |
| | Total | 120 | 87.6 | −27.0 |
| 3000-5000 | Multivehicle | 68 | 53.9 | −14.7 |
| | Single vehicle | | | |
| | Run-off-road and fixed-object | 67 | 52.9 | −21.4 |
| | Other | 29 | 36.7 | +26.6 |
| | Total | 164 | 143.5 | −12.5 |
| 5000-7000 | Multivehicle | 27 | 16.9 | −37.4 |
| | Single vehicle | | | |
| | Run-off-road and fixed-object | 12 | 12.0 | 0 |
| | Other | 6 | 8.2 | +36.6 |
| | Total | 45 | 37.1 | −17.6 |

**Table 6. Safety benefits of converting to four-lane poor-boy highway: nonintersection accidents.**

| Traffic Volume | Type of Accident | No. of Accidents Before | After | Change (%) |
|---|---|---|---|---|
| 1000-3000 | Multivehicle | 35 | 44.8 | +28.0 |
| | Single vehicle | | | |
| | Run-off-road and fixed-object | 72 | 69.4 | −3.6 |
| | Other | 33 | 43.4 | +31.5 |
| | Total | 140 | 157.6 | +12.6 |
| 3000-5000 | Multivehicle | 73 | 44.0 | −39.8 |
| | Single vehicle | | | |
| | Run-off-road and fixed-object | 72 | 68.7 | −4.6 |
| | Other | 40 | 37.1 | −7.3 |
| | Total | 185 | 149.8 | −19.0 |
| 5000-7000 | Multivehicle | 53 | 39.6 | −25.3 |
| | Single vehicle | | | |
| | Run-off-road and fixed-object | 55 | 28.8 | −47.6 |
| | Other | 29 | 30.3 | +4.5 |
| | Total | 137 | 98.7 | −28.0 |

interesting to note that the number of single-vehicle accidents did not decrease until the volume reached 5000 vehicles/day. This indicates the point at which the workload on a four-lane highway becomes great enough to capture the driver's attention.

## PAIRED t-TEST COMPARISONS

To supplement the previous analysis, a paired t-test was used to compare accident frequencies for the two years before and the two years after the modification. The data were examined for significant changes in either accident type or severity on a class-by-class basis. A two-tailed t-test at the 90 percent confidence level was used to test for these differences. Findings from this analysis are discussed below.

### All Accidents

Significant differences in before-after accident frequencies for the all-accident data are given in Table 7. When paved shoulders were added to a two-lane roadway, the total number of accidents decreased. This difference was significant in both the high- and low-volume categories. Few changes in the frequency of occurrence for specific types of accidents were noted. The data do indicate that paved shoulders will decrease accident severity on low- and moderate-volume roadways; however, they appear to increase accident severity on high-volume roads. The reasons for this can best be described in the following manner. The average speed on two-lane roadways is not affected by the presence or absence of a paved shoulder until the volume on the facility reaches 5000 vehicles/day. Above this point, the average speed on a roadway with shoulder is at least 10 percent greater than it is on a similar section without shoulder. Previous research (10) has shown that higher speeds are normally associated with increased accident severity. Thus, the benefits that result from a reduction in total accident frequency and a savings in travel time are being partially offset by an increase in the severity of those accidents that do occur.

When a two-lane roadway with paved shoulder was converted to an undivided, four-lane roadway without shoulder, the number of total accidents decreased for roadways with volumes greater than 3000 vehicles/day. However, this difference was significant in the moderate-volume category only. In contrast to the other type of improvement, many changes in the frequency of occurrence of specific types of accidents were noted. As shown, these differences were more common in the moderate- and high-volume categories. Conversion to a poor-boy roadway appears to have an inconsistent effect on accident severity. The frequency with which injury accidents occur increased during the night and decreased during the day. The reasons for these changes are unclear at best; however, the results of the comparative analysis showed that 60 percent of the total accidents on this type of road occur at night. This indicates that darkness may be eliminating visual cues that alert the driver to the closeness of the edge of the roadway.

## Nonintersection Accidents

Significant differences in before-after accident frequencies for the nonintersection accident data are given in Table 8. The addition of paved shoulders caused the total number of accidents to decrease. This difference was significant in all three volume categories. At low volumes, the frequency of several types of single-vehicle accidents

**Table 7. Statistically significant differences in accident types: all accidents.**

| | | 1000-3000 ADT | | 3000-5000 ADT | | 5000-7000 ADT | |
|---|---|---|---|---|---|---|---|
| Type of Conversion | Type of Accident | Change | Time of Occurrence | Change | Time of Occurrence | Change | Time of Occurrence |
| Addition of paved shoulders to two-lane roadway | Angle | + | N | | | | |
| | Head-on | | | | | | |
| | Right-turn | | | | | | |
| | Left-turn | | | | | – | D,T |
| | Rear-end | | | – | D,T | | |
| | Same direction | | | | | – | T |
| | Fixed-object | | | – | N,T | | |
| | Run-off-road | – | D,N,T | | | | |
| | Animal | | | | | | |
| | Other | | | | | | |
| | Total number | – | D,T | | | – | T |
| | PI+F | | | – | N,T | | |
| | Fatal | | | – | N,T | | |
| | Injuries | – | D,T | | | + | D,T |
| | Fatalities | | | – | N,T | + | N,T |
| Conversion of two-lane roadway with paved shoulder to four lanes without shoulder | Angle | | | | | | |
| | Head-on | | | – | D,T | – | D |
| | Right-turn | | | | | + | N |
| | Left-turn | – | D,T | – | D,T | | |
| | Rear-end | | | | | – | D |
| | Same direction | | | | | | |
| | Fixed-object | – | D,T | | | | |
| | Run-off-road | | | – | T | – | D,T |
| | Animal | | | – | N,T | – | N,T |
| | Other | + | D | | | + | T |
| | Total number | | | – | D,T | | |
| | PI+F | + | N | + | N | – | D,T |
| | | – | D | – | D | | |
| | Fatal | | | | | | |
| | Injuries | + | N | + | N | | |
| | | – | D | – | D | | |
| | Fatalities | | | | | – | D |

Note: + = increase, – = decrease, N = nighttime, D = daytime, and T = total.

**Table 8. Statistically significant differences in accident types: nonintersection accidents.**

| | | 1000-3000 ADT | | 3000-5000 ADT | | 5000-7000 ADT | |
|---|---|---|---|---|---|---|---|
| Type of Conversion | Type of Accident | Change | Time of Occurrence | Change | Time of Occurrence | Change | Time of Occurrence |
| Addition of paved shoulders to two-lane roadway | Angle | | | | | | |
| | Head-on | | | | | | |
| | Right-turn | | | | | | |
| | Left-turn | | | – | D,T | – | D,T |
| | Rear-end | | | – | D,T | – | T |
| | Same direction | | | | | – | T |
| | Fixed-object | – | D | | | | |
| | Run-off-road | – | D,N,T | | | | |
| | Animal | – | T | + | N,T | | |
| | Other | | | | | + | T |
| | Total number | – | D,N,T | + | N | – | T |
| | | | | – | D,T | | |
| | PI+F | | | – | T | | |
| | Fatal | | | – | N,T | | |
| | Injuries | – | T | | | – | D |
| | Fatalities | | | – | N,T | + | N,T |
| Conversion of two-lane roadway with shoulder to four lanes without shoulder | Angle | | | | | | |
| | Head-on | | | – | D,T | | |
| | Right-turn | | | | | | |
| | Left-turn | – | D,T | – | D,T | | |
| | Rear-end | | | – | D,T | – | N,T |
| | Same direction | + | T | | | | |
| | Fixed-object | | | | | – | D |
| | Run-off-road | | | | | – | D,T |
| | Animal | | | – | N,T | – | N,T |
| | Other | + | D,T | + | N,T | + | T |
| | Total number | + | T | – | D,T | – | D,T |
| | PI+F | | | | | – | D,T |
| | Fatal | | | | | | |
| | Injuries | + | N | | | – | D |
| | Fatalities | | | | | | |

Note: – = decrease, + = increase, D = daytime, N = nighttime, and T = total.

decreased. At moderate and high volumes, the frequency of several types of multivehicle accidents decreased. This reinforces the premise that shoulders are effective in reducing the occurrence of different types of accidents at the various volume levels. These data also indicate an overall decrease in accident severity when shoulders were added to two-lane roads; however, the number of fatalities did increase in the high-volume category. Again, this increase is probably the result of higher speeds after the modification.

The conversion of a two-lane roadway with paved shoulder to an undivided, four-lane roadway without paved shoulder resulted in an increase in the total number of accidents at low volumes and a decrease at moderate and high volumes. The frequency of occurrence for the various accident types exhibits similar characteristics. The number of injury accidents increased at low volumes and decreased at high volumes. It is interesting to note that the increase occurred at night and the decrease occurred during the daytime. Again, this observation points to a potential nighttime safety problem.

## CONCLUSIONS

Based on the findings from the comparative before-after analysis, several conclusions can be drawn. These involve changes in both accident frequency and accident type after modification of two types of rural Texas highways, as discussed below.

### Addition of Shoulders

The addition of full-width paved shoulders to a two-lane roadway was effective in reducing the total number of accidents that occurred. The magnitude of the reduction and the characteristics of the accidents varied with the traffic volume. These changes were similar for both the all-accident and the non-intersection-accident data bases. At low volumes, the addition of shoulders resulted in fewer single-vehicle accidents (run-off-road and fixed-object). Thus, the shoulder provides additional paved recovery area for drivers inadvertently exiting from the travel way. These results should be expected, since at low traffic volumes the potential is low for multivehicle accidents and high for driver boredom. At moderate volumes, the addition of shoulders reduced the total number of accidents and the severity of those that did occur. Both single-vehicle and multivehicle accidents decreased in number. This indicates that shoulders are being used for accident avoidance as well as recovery maneuvers. On high-volume roadways, these improvements resulted in fewer total accidents; however, they increased the severity of those that did occur. This increase is attributed to increased operating speeds after the shoulder was added to roadways in this volume category.

### Poor-Boy Roadways

When two-lane roadways with paved shoulders were converted to undivided, four-lane roadways without shoulders, the results varied with the volume of traffic. At low volumes, the total accident frequency actually increased after the conversion. At moderate- and high-volume locations, poor-boy roadways resulted in fewer total accidents. The magnitude of the reduction increased with increasing volumes. This type of modification appears to have an inconsistent effect on accident severity. The frequency with which injury accidents occur increases during the night and decreases during the day. These results indicate that darkness may be eliminating visual cues that alert the driver to the hazards associated with this type of roadway.

## SUMMARY OF FINDINGS AND RECOMMENDATIONS

The findings and recommendations drawn from the analysis are summarized below:

1. Addition of full-width paved shoulders to a two-lane roadway is effective in reducing the total number of accidents that occur.
2. Conversion of a paved shoulder to an additional travel lane probably should not be considered unless the volume on the roadway exceeds 3000 vehicles/day.
3. The potential nighttime safety improvements resulting from improved edge-line delineation systems on poor-boy highways should be evaluated.
4. The potential safety improvements resulting from the addition of full-width paved shoulders at major intersections on two-lane roads without shoulders should be evaluated.

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The contents of this paper reflect our views, and we are responsible for the facts and accuracy of the data presented. The contents do not necessarily reflect the official views or policies of TSDHPT or FHWA. This report does not constitute a standard, specification, or regulation.

## REFERENCES

1. M.S. Raff. Interstate-Highway Accident Study. HRB, Bull. 74, 1953, pp. 18-45.
2. E.T. Perkins. Relationship of Accident Rate to Highway Shoulder Width. HRB, Bull. 151, 1957, pp. 13-14.
3. J.C.H. Woo. Correlation of Accident Rates and Roadway Factors. Purdue Univ., West Lafayette, IN, Joint Highway Research Project C-36-59C, July 1957.
4. E.A. Rinde. Accident Rates Versus Shoulder Widths. California Department of Transportation, Sacramento, Rept. CA-DOT-TR-3147-1-77-01, Sept. 1977.
5. R.W. Sanderson. Analysis of Accident Experience Versus Shoulder Width. Transport Canada, Ottawa, Ontario, Tech. Memorandum, April 1977.
6. C.V. Zegeer and J.G. Mayes. Cost-Effectiveness of Lane and Shoulder Widening on Rural, Two-Lane Roads in Kentucky. Kentucky Department of Transportation, Frankfort, Res. Rept. 999, 1979.
7. C.L. Heimbach, W.W. Hunter, and G.C. Chao. Paved Highway Shoulders and Accident Experience. Transportation Engineering Journal, Proc., ASCE, Vol. 100, No. TE4, Nov. 1974, pp. 889-908.
8. D.B. Fambro, D.S. Turner, and R.O. Rogness. Operational and Safety Effects of Driving on

Paved Shoulders in Texas. Texas Transportation Institute, Texas A&M Univ., College Station, Final Rept. 265-2F, Jan. 1981.
9. D.S. Turner, D.B. Fambro, and R.O. Rogness. Effects of Paved Shoulders on Accident Rates for Rural Texas Highways. TRB, Transportation Research Record 819, 1981, pp. 30-37.
10. D. Solomon. Accidents on Main Rural Highways Related to Speed, Driver, and Vehicle. Bureau of Public Roads, U.S. Department of Commerce, 1964.

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*Abridgment*

# Guidelines for Treatment of Right-Turn Movements on Rural Roads

**BENJAMIN H. COTTRELL, JR.**

**A survey of state departments of transportation conducted to identify the criteria currently used in selecting road designs to accommodate right-turning vehicles on rural roads revealed that the decisions are based primarily on judgment. The survey also identified factors to be considered in establishing criteria. Field work identified the range of traffic and roadway conditions encountered at selected rural locations and the effectiveness of the treatments. Guidelines were developed through an analysis of survey responses, field data, and judgment. They are based on the peak-hour (or design-hour) volumes for right-turning traffic and total traffic on the approach to the right-turn treatment. Guidelines are available for two- and four-lane roadways. Other factors to be considered are noted.**

The objective of the research reported in this paper was to develop guidelines for the treatment of right-turn maneuvers on rural roads that would be applicable for a wide range of conditions at intersections. The treatments considered were a radius, a 150-ft taper, and a full-width turn lane. The volumes and speeds of right-turning and through traffic were the primary factors considered, and the research was limited to treatments for nonsignalized intersections. Information on which to base the guidelines was obtained from a survey of state departments of transportation, conversations with traffic engineers in Virginia, and studies of selected rural intersections.

## SURVEY OF STATE DEPARTMENTS OF TRANSPORTATION

The survey of state departments of transportation (DOTs) was conducted by telephone. If a policy or procedure was in use, a written document was requested. Responses were obtained from 41 of the 48 contiguous states. Of the 25 states without criteria, most consider special right-turn treatment on a project-by-project basis. Whereas several states seldom consider special treatment for right turns in rural areas, 39 percent, or 16, of the state DOTs contacted used some form of criteria. Five guidelines were based on volume conditions, 4 on roadway type, 2 on capacity, and 5 on rule of thumb at intersections. The guidelines should provide efficient use of treatments for right-turn movements and consistent treatment of right-turn movements throughout the state.

Based on the literature review and survey, the following parameters were selected for consideration in the guidelines: (a) total or through traffic volume, (b) right-turn traffic volume, (c) speed prior to the intersection, (d) traffic conflicts due to right-turning vehicles, (e) capacity analysis, and (f) accident history.

## FIELD WORK AND ANALYSIS

The traffic data were collected in two stages: a 48-h count and two 2-h peak-period observations. For the 48-h traffic count, counters were placed prior to the intersection for total volume counts on the approach to the study site and at the intersection for right-turn volume counts. The average daily traffic count and peak 2-h period were determined through a computer analysis. In the next stage, observations were made during two 2-h peak periods to obtain volume counts for all approaches, traffic conflicts due to right-turning vehicles, and speed data on the study approach. Data were collected over 15-min intervals by using a procedure developed by Glauz and Migletz (1).

Twenty-one sites were selected under three classifications. Eight sites were four-lane arterials intersecting two-lane roads, 8 were intersections of two-lane arterials and two-lane roads, and 5 were intersections of two secondary roads. There were 7 sites for each right-turn treatment. There were 11 T-intersections and 10 cross (or four-legged) intersections. There was variability in the lengths and widths of right-turn treatments, and the minor roadway was controlled by a stop sign.

The data analysis used the Statistical Package for the Social Sciences (2) and consisted of two stages: (a) the Pearson correlation to identify parameter pairs that were strongly related and (b) a regression analysis to define the linear relations between these pairs. The study sites were grouped in three ways: by site classification, by type of right-turn treatment, and by right-turn treatment and number of lanes on the major approach. The third grouping was most useful in developing the guidelines.

The analyses indicated that the strongest correlations were between peak-hour-volume right-turn conflicts (PHV conflict) and peak-hour-volume percentage of right turns (PHV % right turns) and between PHV conflict and PHV right turns. The peak-hour period was selected because it is the recommended design period in the American Association of State Highway Officials "Blue Book" (3). There was a strong interest in using PHV total and right-turn volumes. For existing intersections, the use of conflicts requires trained personnel and man-hours for observation, whereas volume counts require only mechanical counters. For proposed intersection sites, an estimate of conflicts adds uncertainty, since it would be based on forecast volume data. However, the correlations were not as strong for PHV total and PHV right turn. For these reasons, no clear-cut guidelines resulted from the field data.

GUIDELINES FOR RIGHT-TURN MOVEMENTS

In view of the above, guidelines were developed by using a synthesis of the field data, guidelines of state DOTs, and engineering judgment. The field data provided the basic framework for the guidelines, but the standards used by other state DOTs, especially Iowa and Idaho, were strong influences. Finally, where the first two items were insufficient, engineering judgment was used.

An explanation of how this was accomplished for two-lane highways is given below. Figure 1 shows the regression lines for the radius and lane treatments and the positions of the study sites, where R, L, and T indicate radius, lane, and taper sites, respectively. The $R^2$ value for the radius line is 0.6 and that for the lane line is 0.2. Since the $R^2$ value for the taper line was less than 0.1, this line was not used. The area below the radius suggests a radius treatment, the area between the two lines suggests a taper treatment, and the area above the lane line suggests a lane treatment.

When the guidelines of state DOTs and judgment were used, the guidelines for right-turn movements took the form of Figure 2. The taper range was expanded on the Y-axis by using data from other guidelines, and the lines were leveled off at the points of maximum total PHV for the field sites. The volume conditions for the respective treatments for two-lane highways are indicated.

It was noted that there were more PHV right turns on highways with speed limits under 55 mph. These roads had radius treatments and residential or commercial development close to the roadside without available right-of-way for any special treatment. An adjustment was needed to accommodate these sites effectively in the guidelines.

For two-lane highways with posted speeds of 45

Figure 1. Development of guidelines for right-turn treatment: two-lane road.

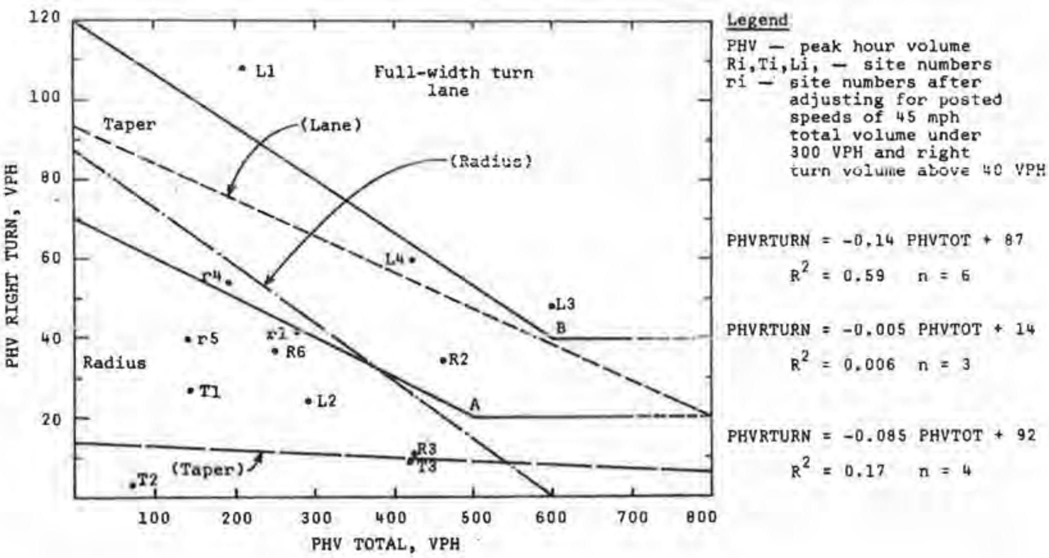



Figure 2. Guidelines for right-turn treatment: two-lane highway.

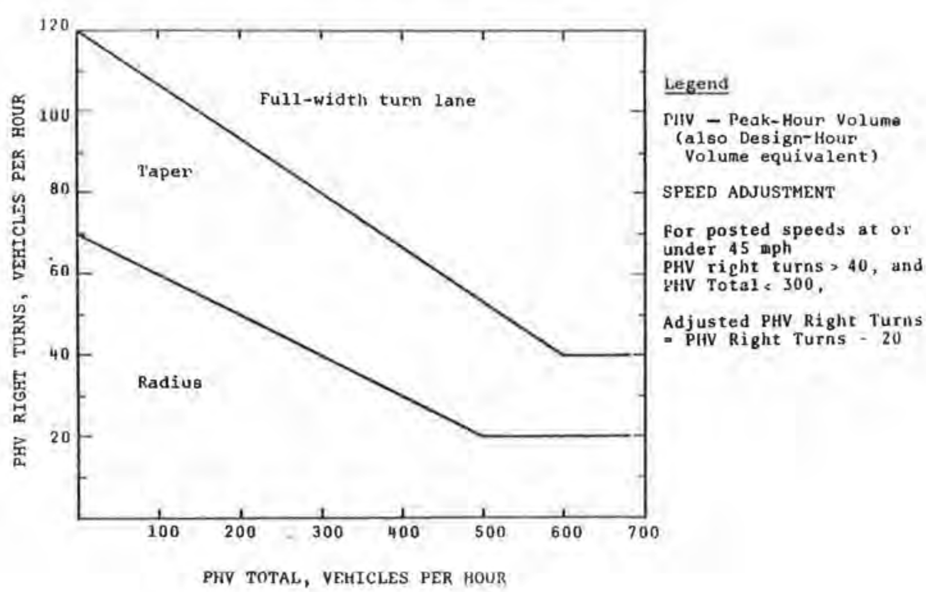
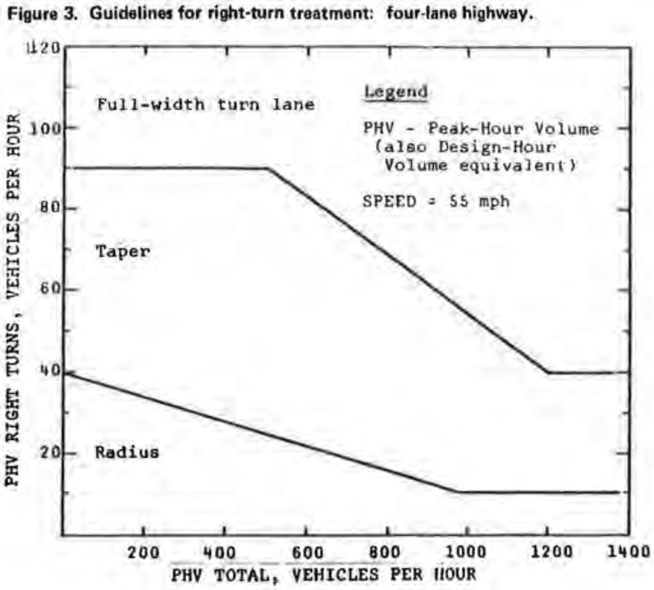


Figure 3. Guidelines for right-turn treatment: four-lane highway.




mph or less, more than 40 PHV right turns, and PHV total of less than 300, the adjusted number of PHV right turns = PHV right turns - 20.

The guidelines for four-lane highways were developed in a similar manner and are shown in Figure 3. These high-level-of-service facilities were divided highways with 55-mph speed limits.

CONCLUSIONS

Although the original intent of the study was to eliminate judgment in developing the guidelines, this could not be done where field data were lacking. The synthesis approach placed emphasis on the field data.

The guidelines are to be used as an aid in the selection of right-turn treatments for new facilities based on forecast demand and for intersection improvements. Site-specific factors of concern that were not addressed are sight distance, grade, availability of right-of-way, and angle of turn. It is suggested that methods that reflect the special concerns be used in lieu of the guidelines for these cases. It is important that this sort of flexibility be a part of the guidelines.

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The opinions, findings, and conclusions are mine and not necessarily those of the sponsoring agencies.

REFERENCES

1. W.D. Glauz and D.J. Migletz. Application of Traffic Conflict Analysis at Intersections. NCHRP, Rept. 219, 1980.
2. N.H. Nie and others. Statistical Package for the Social Sciences, 2nd ed. McGraw-Hill, New York, 1975.
3. A Policy on Geometric Design of Rural Highways. American Association of State Highway Officials, Washington, DC, 1965.

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*Abridgment*

# Motorist Response to Selected Driveway Systems

STEPHEN H. RICHARDS AND CONRAD L. DUDEK

**The results of a human-factors laboratory study conducted at a shopping mall in Bryan, Texas, to evaluate the influence of driveway layout on driveway selection by motorists are presented. The 200 licensed drivers who participated in the studies were shown one of four driveway layouts and asked a series of questions concerning use of the driveway(s) in the layout. The studies revealed that the physical layout of driveways can influence motorists' expectations and interpretation of traffic operations at the driveways. In particular, motorists will perceive that certain driveways are one-way and others are two-way, depending on the physical layout. Certain driveway layouts also imply to some motorists that particular maneuvers (such as left-turn exit or left-turn entry) are prohibited. The studies also found that individual drivers may interpret and respond to particular driveway layouts differently. Most motorists, however, are very reluctant to violate the basic premise of traffic flow in the United States—i.e., keep to the right. In terms of driveway operations, this means that motorists will tend to use driveways that they perceive to be to their right.**

Much attention has been given to the design and operation of individual driveways. All states and most cities closely regulate the design and operation of individual driveways in the interest of improved traffic safety and flow (1). In most cases, however, these regulations do not specifically address the fact that most driveways are part of a "driveway system", or a group of driveways serving the same land development (2), and, accordingly, what happens at one driveway will influence operations at all other driveways in the system. Thus, more emphasis should be given to a "systems approach" in designing and operating driveways. Unfortunately, very little is known about how particular driveway systems are perceived by motorists and how these systems perform.

HUMAN-FACTORS STUDY

A human-factors study was developed to investigate the influence of driveway system layout on driveway selection by entering and exiting motorists. The study was conducted at a regional shopping mall in Bryan, Texas (Bryan has a metropolitan area population of approximately 100 000). Some 200 licensed drivers participated in the study. The participating drivers were younger and better educated, on the average, than drivers as a whole.

In the study, drivers were shown one of the four slides shown in Figure 1. On seeing the slide, they were asked which driveway they could and would use to perform various maneuvers (left-turn entry, right-turn exit, etc.). Note from Figure 1 that the site layout in all of the slides was identical. They all showed the same grocery store, parking lot, and street from the same perspective. The only feature that changed from slide to slide was the driveway layout.

Slide 1 in Figure 1 shows three driveways into the grocery-store parking lot. All three driveways are perpendicular to the roadway and have identical widths and curb return radii. The driveway system in slide 1 depicts a driveway configuration in which all the driveways are equally attractive to a motorist entering or leaving the parking area (except for their relative location).

Slide 2 in Figure 1 shows a dual-driveway system at the grocery store. Both of the driveways are perpendicular to the roadway and are identical in width and curb radii. This driveway system was designed to present a situation that may imply one-way operation.

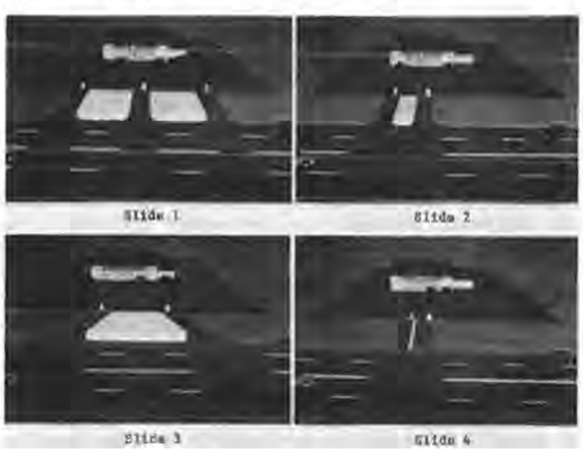
Slide 3 in Figure 1 again shows a dual-driveway system; however, the driveways are not perpendicular to the roadway. Instead, they meet the street at 60° and 120° angles, respectively. This driveway system is intended to present a driveway configuration in which one-way operation may be implied and some turning maneuvers may be discouraged.

Slide 4 in Figure 1 shows a configuration similar to the one in slide 2. However, the spacing between driveways has been decreased so that the system resembles a single large driveway with a raised median in the middle to separate entering and exiting traffic.

STUDY RESULTS

The results of the human-factors studies are summarized in Table 1. Each section of the table pertains to one of the four driveway layouts evaluated (Figure 1). The table gives the percentages of subject drivers who indicated they "could use" certain driveways (question a) and "would use" a particular driveway (question b) to perform four basic driveway maneuvers. The four basic maneuvers--left-turn entry, right-turn exit, right-turn entry, and left-turn exit--are illustrated in Figure 2.

Figure 1. Driveway systems evaluated in the study.




Three-Driveway System

For the three-driveway configuration (slide 1), more than two-thirds of the drivers sampled said that they could use any of the three driveways to enter or exit the grocery-store parking lot. From the table, 70, 74, 68, and 66 percent of the drivers said that they could use driveways A, B, and C for the left-turn entry, right-turn exit, right-turn entry, and left-turn exit maneuvers, respectively. Thus, most of the drivers who participated in the study interpreted the three-driveway layout as a group of three two-way driveways.

Not all of the drivers, however, agreed with this interpretation. A significant portion of them said that only driveways B and C could be used to enter the parking area and only driveways A and B could be used to exit the parking area. These motorists, based on their responses, may have thought that driveways A and C were operating as a one-way pair (entry at driveway A and exit at driveway C) and that two-way operation was permitted at driveway B.

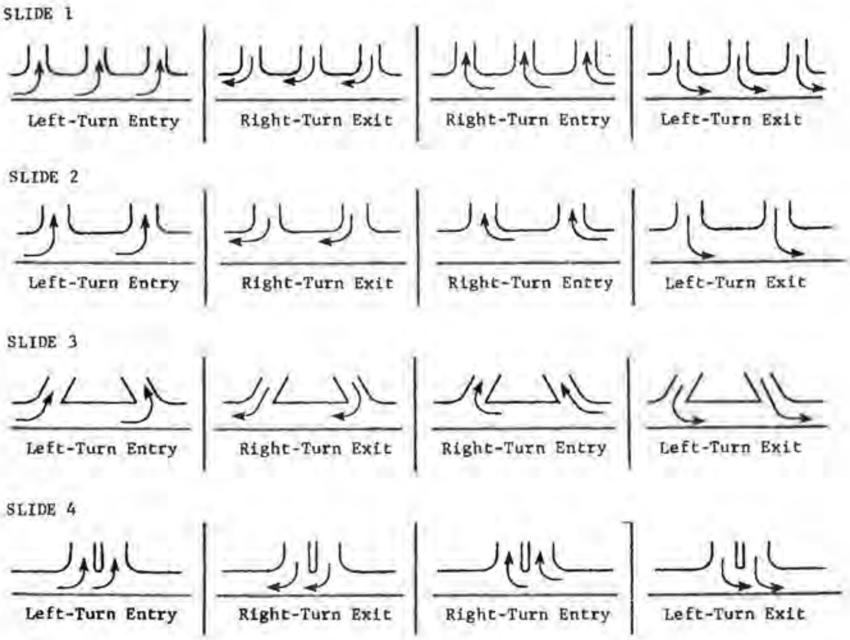
In response to question b, the data in the table indicate that most drivers said they would use the driveway nearest their origin when making a right-turn entry or exit maneuver. For example, 94 percent of the drivers said they would use driveway C for the right-turn entry maneuver. Driveway C is the first driveway at which a motorist would arrive if traveling to the store.

Table 1. Percentage of drivers who selected various maneuvers for each driveway layout in response to questions a and b.

| Response | Driveway | Left-Turn Entry | Right-Turn Exit | Right-Turn Entry | Left-Turn Exit |
|---|---|---|---|---|---|
| Slide 1[a] | | | | | |
| Could use driveway | A only | 2 | 4 | 0 | 0 |
| | B only | 4 | 0 | 0 | 2 |
| | C only | 0 | 0 | 6 | 6 |
| | A and B | 6 | 18 | 0 | 2 |
| | A and C | 2 | 4 | 0 | 0 |
| | B and C | 16 | 0 | 26 | 24 |
| | A, B, and C | 70 | 74 | 68 | 66 |
| Would use driveway | A | 35 | 92 | 0 | 12 |
| | B | 41 | 8 | 6 | 40 |
| | C | 24 | 0 | 94 | 48 |
| Slide 2[b] | | | | | |
| Could use driveway | A only | 8 | 64 | 8 | 64 |
| | B only | 64 | 8 | 64 | 8 |
| | A and B | 28 | 28 | 28 | 28 |
| Would use driveway | A | 14 | 88 | 12 | 86 |
| | B | 86 | 12 | 88 | 14 |
| Slide 3[a] | | | | | |
| Could use driveway | A only | 2 | 68 | 2 | 68 |
| | B only | 68 | 2 | 68 | 2 |
| | A and B | 30 | 30 | 30 | 30 |
| Would use driveway | A | 14 | 98 | 2 | 86 |
| | B | 86 | 2 | 98 | 14 |
| Slide 4[c] | | | | | |
| Could use driveway | A only | 0 | 100 | 0 | 98 |
| | B only | 100 | 0 | 100 | 2 |
| | A and B | 0 | 0 | 0 | 0 |
| Would use driveway | A | 0 | 100 | 0 | 98 |
| | B | 100 | 0 | 100 | 2 |

[a]Sample size = 50. [b]Sample size = 49. [c]Sample size = 51.

**Figure 2. Basic maneuvers for four driveway layouts studied.**




The consistency among the subjects in driveway selection for the right-turn entrance and exit maneuvers is attributed to two factors:

1. The drivers selected the driveway most convenient in terms of location (the driveway best positioned in terms of where they had come from or where they were going).
2. The drivers' selections were consistent with the normal one-way operation pattern (i.e., enter right and exit left).

Driveway selection trends for the left-turn maneuvers differed significantly from those observed for the right-turn maneuvers. The difference is attributed to the fact that, for left-turn maneuvers, selecting a driveway convenient in terms of location violated the normal one-way operation pattern. For example, 35 percent of the drivers said they would use driveway A, 41 percent said they would use driveway B, and 24 percent said they would use driveway C for the left-turn entry maneuver. Based on these data, it is apparent that many drivers were hesitant to challenge the one-way operational premise (keep to the right) even though there was not an obvious one-way driveway layout shown. Instead, many of the drivers opted for the middle driveway, which was generally acknowledged as a two-way driveway.

Dual-Driveway System

Sixty-four percent of the drivers associated the dual-driveway system presented in slide 2 with normal one-way driveway operation. Most of the remaining drivers (28 percent) indicated that two-way operation was permitted at both driveways. Eight percent of the drivers said that the layout implied a "reverse" one-way driveway operation--in other words, keep to the left. The operational patterns implied to motorists were consistent for the various maneuvers.

As seen in the table, driver responses to question b supported the trends observed in the question a responses. Generally speaking, almost 90 percent of the drivers said they would use the system shown in slide 2 as a one-way driveway pair, whereas slightly more than 10 percent said they would violate the normal one-way operation pattern.

Angle Driveway System

Based on question a responses in the table, 68 percent of the drivers associated the angle driveway system in slide 3 with normal one-way driveway operation. Thirty percent said that all entry and exit maneuvers could be made at both driveways, and only 2 percent said that the layout implied a re-

Driver responses to question b support the trends observed in question a responses, which indicates that most drivers would prefer to use the driveways as a normal one-way pair. Also interesting to note from the question b responses is the fact that normal one-way operation was implied to more drivers for right-turn maneuvers than for left-turn maneuvers (98 versus 86 percent). This result is attributed to the geometric design of the driveways. For example, drivers making a left turn into driveway B must turn sharply because of the angle of the driveway and the relatively small curb return radius. Thus, some drivers were apparently discouraged from using driveway B and encouraged to use driveway A for left-turn entry maneuvers.

Divided-Driveway System

From Table 1, it is apparent that the divided-driveway layout in slide 4 strongly implied normal one-way driveway operation. Only 2 percent of the drivers (1 driver in the sample of 50) said he could and would use driveway B for the left-turn exit maneuver. All other drivers said that they would enter the parking lot by using driveway B and leave the parking lot by using driveway A.

SUMMARY

A human-factors laboratory study was conducted in Texas to evaluate the influence of driveway layout and its relation to other physical features on driveway selection by motorists. Two-hundred motorists participated in these studies. The study results should be regarded as preliminary, since the study was conducted in only one part of the country and the study sample was younger and better educated than the national driving population.

Some of the more significant findings from the studies are summarized below:

1. The physical layout of driveways can influence motorists' expectations and interpretations of traffic operations. Motorists will perceive that certain driveways are one-way and others are two-way, depending on the physical layout. Certain driveway layouts also imply to some motorists that particular maneuvers (such as left-turn exit or left-turn entry) are prohibited.

2. Individual drivers may interpret and respond to a particular driveway layout differently. For example, almost all drivers agreed in their interpretation of a divided-driveway layout, but there was considerable inconsistency among drivers in interpreting and responding to a three-driveway layout.

3. Motorists' reluctance to violate the principle of one-way traffic operation (keep to the right) greatly influences their interpretation of and response to various driveway layouts.

4. Even though one-way operation was strongly implied by all of the two-driveway systems studied, not all motorists agreed with this interpretation. Therefore, it should be recognized that driveway layout alone may not provide enough information for drivers to determine with consistency the intended operation at one-way driveways. Thus, the use of effective signs and markings at these driveways is encouraged.

5. In the studies, drivers said that they would use the driveway closest to their origin (for entry maneuvers) or destination (for exit maneuvers), provided use of that driveway did not violate the normal one-way operation pattern. If it did violate the pattern, however, many drivers said that they would use a different driveway, one whose use was consistent with the normal one-way operation pattern.

6. The angled dual-driveway system implied normal one-way operation to a slightly higher percentage of drivers than the parallel dual-driveway layout. The studies did not fully indicate that angled driveways might also discourage left-turn maneuvers.

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REFERENCES

1. V.G. Stover, W.G. Adkins, and J.C. Goodknight. Guidelines for Medial and Marginal Access Control on Major Roadways. NCHRP, Rept. 93, 1970.
2. S.H. Richards. Guidelines for Driveway Design and Operation: Volume 2--Technical Report. Texas Transportation Institute, Texas A&M Univ., College Station, Rept. 5183-2, 1980.

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