# GUIDELINES FOR SIGNING AND MARKING OF LOW-VOLUME RURAL ROADS

NED E. WALTON, JOHN M. MOUNCE, and WILLIAM R. STOCKTON
Research Engineers
Texas Transportation Institute
Texas A&M University
College Station, Texas

## ABSTRACT

Existing standards and guidelines for the application of signs and markings are unsuited and inefficient for use on low-volume rural roads (less than 400 ADT). To alleviate this inadequacy, several potentially hazardous situations were evaluated to ascertain actual needs for signs and markings as they relate to economy and safety. These evaluations were based on recent research and on probability of conflict analyses regarding the needs for signing and marking of intersections, horizontal curves, and sections of inadequate passing sight distance.

The research revealed that more efficient intersection control can be attained from the careful application of STOP signs and CROSS ROAD warning signs based on approach speed, sight distance, and combined intersecting volumes. It was found that the treatment of horizontal curves can be made more efficient through the application of more stringent guidelines without adversely affecting safety. Striping of no-passing zones was found to be very inefficient in most instances, as the probability of conflict in these situations is virtually nil; guidelines for alternative treatments are presented. Overall, it was the opinion of the authors that application of guidelines suited to the rural context would result in savings in time, money, and frustration on the part of responsible agencies.

## INTRODUCTION

Low-volume rural roads (less than 400 ADT) comprise the bulk of the public roadways operated in this country. Their existence is essential to the various aspects of rural life. Farm-to-market and country roads provide access to the rural communities as well as performing as the major avenues of agricultural commerce. Forest and park roads are necessary for the operation, maintenance, and accessibility of national forests and parks.

Heretofore, the application of traffic control devices on rural roads has been restricted to those guidelines and regulations contained in the *Manual on Uniform Traffic Control Devices* (MUTCD). However, it is easily recognizable that those guidelines, which were developed primarily for major highways and city streets, are impractical for application on low-volume rural roads. Not only is the adherence to existing MUTCD guidelines unnecessarily expensive, but produces considerable visual clutter in the rural environment. Therefore, a reduction in the levels of signing and marking on low-volume rural roads has been given careful consideration.

Contained herein are the guidelines developed for the application of warning and regulatory signs on low-volume rural roads and the analyses that led to their development.

Of primary importance in the reduction of the level of signing and marking is the corresponding effect on safety. To assess this effect, three major situations of potential hazard were analyzed—intersections, horizontal curves, and sections of insufficient passing sight distance. Two of the situations, intersections and no-passing zones, were analyzed using a probability-of-conflict technique. Safety on horizontal curves was based on research by Ritchie, *et. al.,* and field observations made during the course of this research.[2]

One of the overriding concerns throughout the conduct of the research was development of guidelines that were not only easily understood and readily implementable, but were truly suited to the rural situation. Guidelines contained in the MUTCD may result in too little intersection control and too much horizontal curve and no-passing zone warning if applied in the rural context. Therefore, a combination of economic analysis, engineering judgment, and field observation was applied to produce the guidelines. The analyses presented are abridgements of the actual research. Detailed descriptions of the research may be obtained from the Texas Transportation Institute.

## INTERSECTION CONTROL

The analyses and guidelines developed for treatment of low-volume rural intersections stemmed from the question: "What is the probability of the occurrence of an accident at a low-volume rural intersection?"

*Analysis*

The initial step in determining the probability of an accident was the determination of the probability-of-conflict. From this determination, the expected number of accidents per year can be estimated.

For the purpose of analysis, the following assumptions are made:

1. Conflict is defined as that maneuver of vehicle B such that the driver of vehicle A must change speed or direction to maintain a comfortable clearance interval.
2. Assumed average speed is 64 kph (40 mph) or approximately 18 mps (60 fps), and no intersection control or signing is provided.
3. Any two vehicles approaching the intersection from conflicting directions such that the second vehicle would enter the intersection within three seconds after the first vehicle enters the intersection are said to be in conflict; i.e., one or both vehicles must take a speed change maneuver to provide comfortable clearance.
4. Effects of sight distance are not considered in the analysis portion.
5. All vehicles arrive during a 12-hour period from 7 a.m. to 7 p.m. (It is probable that all vehicles do not arrive between 7 a.m. and 7 p.m., but since this assumption covers the worst condition, it is used here.)
6. All arrivals are random; that is, they follow a Poisson distribution.
7. Only one arrival per approach is possible during one three-second interval; i.e., all approaches are single lane and all headways are greater than three seconds.
8. The possibility of vehicles arriving on three approaches within a three-second interval is negated as the probability of such an occurrence is a maximum of $2.01 \times 10^{-5}$ for the volumes under consideration.

The probability that two vehicles will be in conflict is the product of the probability that either vehicle is in the conflict region during the interval $\Delta t$ (3 sec):

$$\begin{aligned} P(\text{conflict}) = {} & P(\text{vehicle A in conflict region during } \Delta t) \times \\ & P(\text{vehicle B in conflict region during } \Delta t) \end{aligned}$$

This probability of conflict analysis revealed that, on the average, 0.68 conflicts per day could be expected on two intersecting roadways of 100 ADT each. ADT's were incremented by 25 vpd on each facility to provide an expected number of conflicts, E(C), for all ADT combinations up to 400 by 400 (800 ADT combined intersecting volumes). Expected number of conflicts ranged from 0.04 per day for a combined ADT of 50 vpd (25 by 25) to 10.67 per day for a combined ADT of 800. Selected values for E(C) shown in Table 1 reveal that the highest expected number of conflicts for a given combined ADT occurs when the intersecting volumes are approximately equal. This indicates that the worst-case condition may not be the intersection of a minor road with a major road, but actually the intersection of two very similar roads.

Given, then, the expected number of conflicts, what is the probability of an accident? Data from a study by Perkins, *et. al.,* indicated about 33 accidents occur in every 100,000 conflicts for the situation in question:

Probability of an Accident, given a Conflict [P(A,C)] = .00033[3]

Other data indicated that P(A,C) ranges from .00025 to .00035. Therefore, to examine worst-case conditions, a value of P(A,C) = .00035 was chosen. Then, the probability of an accident, P(A), is given by:

$$P(A) = P(A,C)\ P(C)$$

Multiplying the probability of an accident occurring in a given three-second interval by the number of such intervals in a day yields the expected number of accidents per day, and thus by 365 yields the expected number of accidents per year, E(A).

TABLE 1—EXPECTED NUMBER OF CONFLICTS PER DAY E(C)

| ADT—Facility B | ADT—Facility A: 100 | 200 | 300 | 400 |
|---|---|---|---|---|
| 100 | .68 | 1.36 | 2.03 | 2.70 |
| 200 | 1.36 | 2.70 | 4.04 | 5.37 |
| 300 | 2.03 | 4.04 | 6.04 | 8.03 |
| 400 | 2.70 | 5.37 | 8.03 | 10.67 |

For the two intersecting facilities of 100 ADT each, E(A) = 0.087. From the selected values of E(A) shown in Table 2, it can be seen that one or more accidents per year can be expected above a combined ADT of approximately 700 vpd.

However, the absolute number of expected annual accidents is not solely important. Of equal or greater importance is the estimated annual cost of accidents in the no-control alternative as it relates to the estimated annual cost of the two-way-stop-control alternative.

Estimated annual cost of accidents at a particular intersection is the product of estimated cost per accident and estimated number of accidents per year. The primary determinant in accident cost is severity. Results of a study by Burke showed little variation in severity over the ADT range 0-400.[4]

However, as would be expected, severity was found to increase with speed[5], as did the proportion of fatalities.[6]

Combining the results of these two studies, a weighted accident cost equation was developed:

$$\text{Cost} = F_p(A) + F_I(B) + F_F(C)$$

where, $F_p$ = proportion of property-damage-only accidents
$A$ = average cost of property-damage-only accidents = \$318[4]
$F_I$ = proportion of injury accidents
$B$ = average cost of injury accidents = \$1955[4]
$F_F$ = proportion of fatal accidents
$C$ = average cost of fatal accidents = \$13,781[4]

TABLE 2—EXPECTED NUMBER OF ACCIDENTS PER YEAR E(A)

| ADT—Facility B | ADT—Facility A: 100 | 200 | 300 | 400 |
|---|---|---|---|---|
| 100 | .087 | .174 | .259 | .345 |
| 200 | .174 | .345 | .516 | .686 |
| 300 | .259 | .516 | .772 | 1.026 |
| 400 | .345 | .686 | 1.026 | 1.363 |

Combining the proportional factor for each type of accident with the average cost of that type of accident in the preceding equation resulted in a weighted average cost per accident for each speed group. For example, the weighted average cost of 32 kph (20 mph) accidents would be found as follows:

$$\text{Cost/Accident (32 kph)} = .750(\$318) + .248(\$1955) + .002(\$13{,}781) = \$750$$

These cost and the proportional factors from which they were derived are shown in Table 3.

TABLE 3—WEIGHTED AVERAGE COST PER ACCIDENT, BY SPEED

| Speed (kph) | Proportional Factors $F_P$ | $F_I$ | $F_F$ | Weighted Average Cost/Accident ($) |
|---|---|---|---|---|
| 32 | .750 | .248 | .002 | 750 |
| 48 | .720 | .277 | .003 | 812 |
| 64 | .660 | .322 | .008 | 969 |
| 80 | .580 | .400 | .020 | 1,242 |
| 96 | .410 | .783 | .077 | 1,733 |

(Note: 1 kph = .625 mph)

Average yearly accident cost per intersection by speed for each ADT combination is given by the product of expected number of yearly accidents, E(A) (Table 2), and weighted average cost per accident (Table 3). These costs were compared with costs associated with the use of two-way-stop control. Two-way-stop control costs included expected accident cost (approximately 20 percent that of no control) and additional annual motor vehicle operating costs due to the stop control.

Additional operating cost is the difference between 1) the cost of continuing through the intersection at the approach speed; and 2) the cost of slowing to a stop from the approach speed and returning to the previous speed. As would be expected, the costs of stopping and regaining running speed increase with higher running speeds. Table 4 shows additional operating costs for each speed group and the compilation of expected cost of two-way-stop control on a 100 ADT facility.

TABLE 4—EXPECTED ANNUAL COSTS ASSOCIATED WITH TWO-WAY-STOP CONTROL

| Approach Speed (kph) | Operating Cost Per Stop (5) ($) | Stops Per Year (100 ADT) | Annual Operating Cost ($) Col. A | Average Cost/Accident ($) | Expected Number of Accidents | Expected Annual Accident Cost ($) Col. B | Expected Annual Cost of Two-Way-Stop ($) Col. A + Col. B |
|---|---|---|---|---|---|---|---|
| 32 | .0022 | 36,500 | 81 | 750 | .0174 | 13 | 94 |
| 48 | .0040 | 36,500 | 145 | 812 | .0174 | 14 | 159 |
| 64 | .0059 | 36,500 | 216 | 969 | .0174 | 17 | 233 |
| 80 | .0083 | 36,500 | 302 | 1,242 | .0174 | 22 | 324 |
| 96 | .0116 | 36,500 | 422 | 1,733 | 0.174 | 30 | 452 |

Source: Reference 7 (Note: 1 kph = .625 mph)

## TABLE 5—EXAMPLES OF ESTIMATED ACCIDENT COSTS PER YEAR

Approach Speed—32 kph (20 mph)

ADT—Facility A

| | 0 | 100 | 200 | 300 | 400 |
|---|---|---|---|---|---|
| ADT—Facility B | 100 | 65 | 130 | 194 | 259 |
| | | *94** | *107* | *120* | *133* |
| | 200 | 130 | 259 | 387 | 514 |
| | | *107* | *213* | *238* | *264* |
| | 300 | 194 | 387 | 579 | 770 |
| | | *120* | *238* | *357* | *395* |
| | 400 | 259 | 514 | 770 | 1022 |
| | | *133* | *264* | *395* | *526* |

*Two-way-stop control costs are shown in italics

Approach Speed—64 kph (40 mph)

ADT—Facility A

| | 0 | 100 | 200 | 300 | 400 |
|---|---|---|---|---|---|
| ADT—Facility B | 100 | $ 84 | $169 | $251 | $334 |
| | | *233* | *250* | *266* | *283* |
| | 200 | $169 | $334 | $500 | $ 665 |
| | | *250* | *499* | *532* | *565* |
| | 300 | $251 | $500 | $748 | $ 994 |
| | | *266* | *532* | *798* | *847* |
| | 400 | $334 | $665 | $994 | $1320 |
| | | *283* | *565* | *847* | *1129* |

Approach Speed—96 kph (60 mph)

ADT—Facility A

| | 0 | 100 | 200 | 300 | 400 |
|---|---|---|---|---|---|
| ADT—Facility B | 100 | 151 | 302 | 449 | 598 |
| | | *452* | *482* | *512* | *542* |
| | 200 | 302 | 598 | 894 | 1189 |
| | | *482* | *965* | *1024* | *1083* |
| | 300 | 449 | 894 | 1338 | 1778 |
| | | *512* | *1024* | *1536* | *1624* |
| | 400 | 598 | 1189 | 1778 | 2362 |
| | | *542* | *1083* | *1624* | *2162* |

Selected values of costs associated with no control and two-way-stop control are compared in Table 5, with two-way-stop control costs shown in italics.

Careful examination of the estimated costs tables reveals that, up to 200 vpd combined volumes, the expected annual accident costs associated with no control are less than the accident and operating costs associated with two-way-stop control. At higher ADT's, these expected costs become equal, and higher still; the no-control alternative becomes more expensive. As a result of increased operating costs with increased running speeds, this breakpoint between the economic justification of the two-control alternatives increases as the speed on the intersecting roadways increases. These analyses showed that the no-control alternative was more economical up to the following combined ADT's:

| Speed (kph) | Combined ADT (vpd) |
|---|---|
| 32 (20 mph) | 300 |
| 48 (30 mph) | 520 |
| 64 (40 mph) | 650 |
| 80 (50 mph) | 700 |
| 96 (60 mph) | 720 |

The calculation of these breakpoints is derived by equating the costs of the no-control alternative and the costs of the two-way-stop control alternative, as represented in the following equation:

$$E(A) \cdot C_A = ADT \cdot 365 \cdot C_S - 0.2\,[E(A) \cdot C_A]$$

which can be simplified to:

$$0.8[E(A) \cdot C_A] = T_Y \cdot C_S$$

where, $E(A)$ = expected number of yearly accidents with no control [for equally split traffic volumes (Table 2)]
$C_A$ = weighted average cost per accident (Table 3)
$T_Y$ = yearly traffic volume = ADT x 365
$C_S$ = additional motor vehicle operating cost with two-way-stop control (Table 4, Column A)

Thus, for each approach speed there is a point below which stop control is not economically justified. However, as mentioned previously, economy is not the only necessary consideration. Although two-way-stop control may not be economically justified, adequate visibility of a crossing roadway is vital in the absence of signing. As it is highly likely that a situation will arise in which stop control is not justified and crossroad visibility is inadequate, a standard CROSS ROAD warning sign (W2-1 in MUTCD) is necessary. Criteria for the use of a crossroad sign was based on sight distance requirements specified by AASHTO.[7]

The inclusion of the CROSS ROAD warning sign as part of low-volume rural intersection control was, in the opinion of the authors, a necessary safety measure in the absence of stop control and adequate sight distance. Although the erection of four CROSS ROAD signs is more expensive than two STOP signs, the savings in motor vehicle operating costs over the life of the signs more than offset the additional capital cost of the CROSS ROAD signs.

*Guidelines*

The above analyses, coupled with engineering judgment and many hours of field observation in rural areas, resulted in the following recommended guidelines for safe and economic low-volume rural intersection control:

STOP signs should be placed on low-volume rural roads (paved or unpaved) intersecting paved highways, provided that the low-volume road meets one or more of the following criteria:

The low-volume road:

a. serves ten or more residences;
b. has an average daily traffic (ADT) of 50 or more; or
c. is 8 kilometers (5 mi) long or longer.

The above guidelines should be followed unless it can be shown that:

1. The combined average daily traffic for the two intersecting roadways is less than that shown below for the corresponding lower approach speed of the two facilities:

| Approach Speed (kph) | Combined ADT |
|---|---|
| 32 (20 mph) | 300 vpd |
| 48 (30 mph) | 500 vpd |
| 64 (40 mph) | 640 vpd |
| 80 (50 mph) | 700 vpd |
| 96 (60 mph) | 720 vpd |

2. The sight distance on each approach is at least that shown below for the corresponding approach speed:

| Approach Speed (kph) | Sight Distance (m.) |
|---|---|
| 32 (20 mph) | 27 ( 90 ft) |
| 48 (30 mph) | 39 (130 ft) |
| 64 (40 mph) | 54 (180 ft) |
| 80 (50 mph) | 66 (220 ft) |
| 96 (60 mph) | 78 (260 ft) |

Sight distance is defined here as a triangle of clear visibility with legs of a length equal to the distance shown for the corresponding speed. This triangle shall apply from all directions of approach.

*Example*: Approach speeds on two intersecting facilities are 80 kph (50 mph) and 64 kph (40 mph), respectively. A driver approaching the intersection on the 80 kph facility must, at a distance of 66 meters (220 ft) from the intersection, have clear visibility throughout a cone of vision extending 54 meters (180 ft) in each direction along the crossing roadway (Figure 1).

For intersections which meet the requirements of (1) above for no control, but do not meet the requirements of (2) above (i.e., inadequate sight distance), a standard CROSS ROAD sign, W2-1, may be used in advance of the intersection in lieu of two-way-stop control.

The requirements for intersection control given above can be determined graphically from Figure 2. The procedure is as follows:

Step 1. Enter combined ADT in part (A) and project horizontally to intersect with lowest approach speed. If the intersection of these two lines is above the curve (shaded area), stop here and install STOP signs on the minor approach(es).

Step 2. If below the curve, project intersection point downward into part (B).

Step 3. Enter shortest sight distance on lower speed approach and project horizontally to intersect line drawn in Step 2. If this intersection point lies below the line, no control is needed. If the intersection point lies above the line (shaded area), a standard CROSS ROAD sign is needed on all approaches.

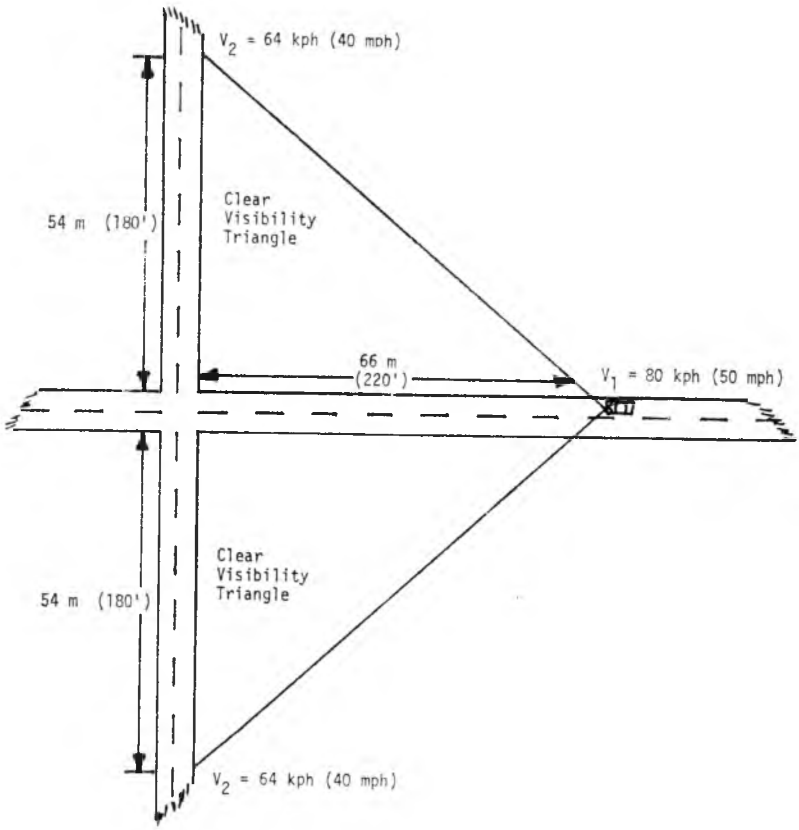


**Fig. 1. Required Sight Distance Triangle for No Intersection Control**

## HORIZONTAL CURVES

Aside from the elements of geometric design, use of warning signs is one of the primary methods of improving safety on horizontal curves. In an effort to provide guidelines for the application of CURVE warning signs on low-volume rural roadways, existing practices, recent research, and subjective data obtained in this study were assimilated. Recommendations based on these elements were developed. Contained herein is the procedure followed in the development of recommendations and guidelines.

### *Analysis*

The *Manual on Uniform Traffic Control Devices* (MUTCD) provides minimal guidelines for the application of CURVE signs and

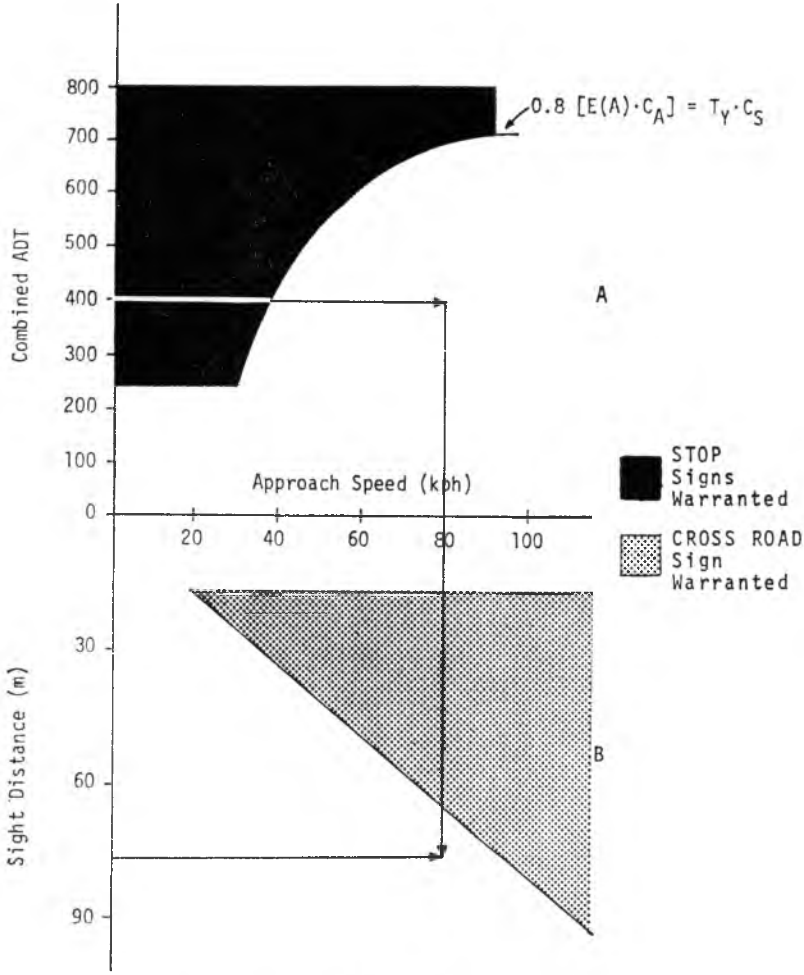


**Fig. 2. Intersection Signing Needs Diagram**
**Note: 1 kph = 0.6 mph, 1 m = 3.3 ft**

advisory speed plates. Several states have developed specific warrants for CURVE signs within the requirements of the MUTCD. These warrants require the availability of ball bank indicators or detailed curve data. The objective of this endeavor was to establish guidelines for curve signing in lay terms to permit ready application.

The primary assumption made was that supplemental driver information (signs, markings, etc.) is more critical in nighttime driving than in daytime. Utilizing the equation

$$S = .277V_1T + \frac{.277^2[V_1^2 - V_2^2]}{2a}$$

where, $S$ = required deceleration distance (meters)
$T$ = perception-reaction time
$V_1$ = approach speed (kph)
$V_2$ = safe curve speed (kph)
$V_3$ = deceleration rate ($mps^2$),

required distances for deceleration to safe curve speed were calculated assuming an average deceleration rate of -2.1 $mps^2$ (-7 $fps^2$). The addition of a perception-reaction time of two seconds yielded the minimum distance at which a driver must be aware of an impending situation. These distances are shown for various combinations of approach and curve speeds in Table 6.

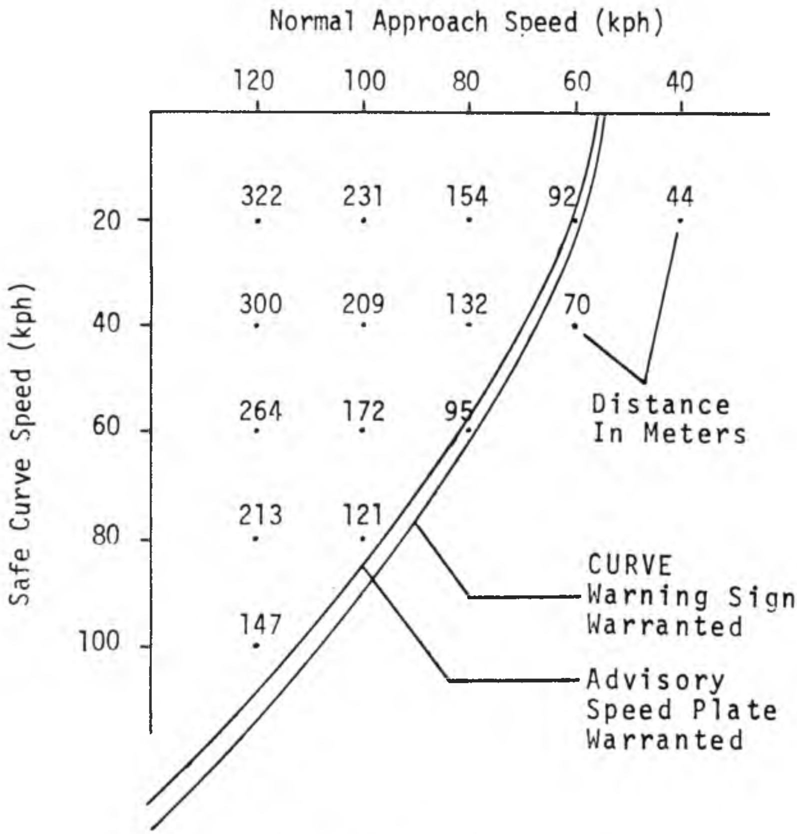


**Table 6. Required Deceleration Distances on Horizontal Curves**
**Note: 1 kph = .625 mph, 1 meter = 3.3 ft**

For certain combinations of approach and curve speed the roadway itself provides, in general, adequate information for proper vehicular maneuvers. It was assumed that high beam visibility distance [about 90 meters (300 ft)] was the upper limit at which the roadway provides adequate information. A line was drawn on Table 6 through the 90-meter contour. Distances to the upper left of the contour line require advance supplemental information, while distances to the lower right do not.

Calculated data points were compared with field observations. A close correlation was found between calculated critical speed differentials and those curves observed to be hazardous.

In general, it was found that at approach speeds greater than 48 kph (30 mph), a differential of 16 kph (10 mph) between approach speed and safe curve speed required perception-reaction-deceleration distances necessitating advance warning. This advance warning can be provided through the use of standard CURVE signs (W2-1 in MUTCD). Speed differentials of 24 kph (15 mph) are characteristic of more severe curvature and should be identified with a CURVE sign (W2-1) and an advisory speed plate (W13-1).

The relative degree of risk associated with this reduced level of signing on curves can be evaluated based on driver characteristics in a curve maneuver. The important question to be answered is whether the reduced level of signing (fewer or no signs) contributes to potentially hazardous operations. To determine the effect of signing level, a study was conducted by Ritchie, *et. al.,* in 1968. Their study involved the relationship between forward velocity and lateral acceleration in curve driving. In a subsequent study, Ritchie expanded the previous research to determine the driver's choice of curve speed as a function of curve and advisory speed signs.[2]

The study was based on the actions of 50 subjects negotiating sections of roadways containing 162 curves which required deceleration from normal operating speed. Four levels of signing were evaluated: (1) no signs; (2) CURVE signs; (3) CURVE signs with advisory speed plaques; and (4) CURVE signs without advisory speed plaques. In addition, all curves were lumped together to obtain an overall condition. The significant results of the study were (Table 7):

1. As forward velocity increased, lateral acceleration decreased, indicating that at higher speeds drivers tend to provide themselves with a greater margin of safety on curves.
2. Drivers were more cautious on curves without signs than on curves with signs. Mean lateral accelerations on curves with

TABLE 7—LATERAL ACCELERATION IN GRAVITATIONAL UNITS (G) AS A FUNCTION OF FORWARD VELOCITY AND TYPE OF ROADWAY SIGN

| | | Forward Velocity (kph) | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Condition | | <32 | 32-40 | 40-48 | 48-56 | 56-64 | 64-72 | 72-80 | 80-88 |
| All Curves | Mean | 0.264 | 0.257 | 0.228 | 0.201 | 0.212 | 0.172 | 0.142 | 0.129 |
| | SD | 0.055 | 0.070 | 0.061 | 0.051 | 0.042 | 0.051 | 0.043 | 0.041 |
| | N | 9 | 6 | 11 | 16 | 20 | 28 | 35 | 37 |
| With Signs | Mean | 0.280 | 0.270 | 0.257 | 0.222 | 0.223 | 0.183 | 0.159 | 0.174 |
| | SD | 0.024 | 0.071 | 0.061 | 0.053 | 0.035 | 0.051 | 0.037 | 0.028 |
| | N | 2 | 5 | 6 | 10 | 13 | 21 | 18 | 4 |
| Without Signs | Mean | 0.259 | 0.193 | 0.193 | 0.165 | 0.192 | 0.140 | 0.124 | 0.124 |
| | SD | 0.062 | 0.000 | 0.043 | 0.021 | 0.048 | 0.139 | 0.042 | 0.040 |
| | N | 7 | 1 | 5 | 6 | 7 | 7 | 17 | 33 |
| With Advisory Speed | Mean | 0.263 | 0.268 | 0.257 | 0.222 | 0.224 | 0.185 | 0.161 | 0.169 |
| | SD | 0.000 | 0.081 | 0.061 | 0.053 | 0.037 | 0.053 | 0.043 | 0.032 |
| | N | 1 | 4 | 6 | 10 | 12 | 19 | 13 | 3 |
| Without Advisory Speed | Mean | 0.264 | 0.234 | 0.193 | 0.165 | 0.195 | 0.146 | 0.130 | 0.126 |
| | SD | 0.059 | 0.059 | 0.043 | 0.021 | 0.045 | 0.035 | 0.039 | 0.041 |
| | N | 8 | 2 | 5 | 6 | 8 | 9 | 22 | 34 |

Source: Reference 2 (Note: 1 kph = .625 mph) signs ranged from 0.280g to 0.159g, and on curves without signs, from 0.259g to 0.124g.

3. Except at very low speeds, greater lateral acceleration (0.268g to 0.161g) was produced on signed curves with advisory speed plaques than on signed curves without advisory speed plaques.
4. Below 64 kph (40 mph), posted advisory speeds were exceeded more often than above 64 kph.

The author's conclusion was that ". . . the experimental data do not support the hypothesis that the roadway signs are responsible for the inverse relationship between speed and lateral acceleration." Roadway signs serve to reduce uncertainty and increase the confidence with which the driver proceeds. Therefore, it is concluded that the reduced level of signing on curves on low-volume rural roads can be effected without appreciable decrease in level of safety.

### *Guidelines*

Based on the foregoing analyses and associated assessment of relative degree of risk, and on engineering judgment founded on field observations, the following guidelines were developed:

CURVE signs (W1-2) should be placed in advance of all curves with intersecting angles of 45 degrees or more on paved roadways, and 60 degrees or more on unpaved roadways unless it can be shown that:

1. the posted speed limit is 55 kph (35 mph) or less; or
2. the combination of normal approach speed and safe curve speed requires a perception-reaction-deceleration distance of less than 90 meters (300 ft); i.e., the combination of the above speeds produces a point to the lower right of the 90-meter contour line in Figure 3.

Advisory speed plates (W13-1) should be used in conjunction with CURVE warning signs when the safe curve speed is 8 kph (5 mph) below that speed warranting a CURVE sign; i.e., to the upper left of the appropriate line in Figure 3.

## NO-PASSING ZONES

As most low-volume rural roads follow the existing horizontal and vertical curvature of the terrain, there can be a considerable amount of inadequate passing sight distance. Treatment of this condition, with

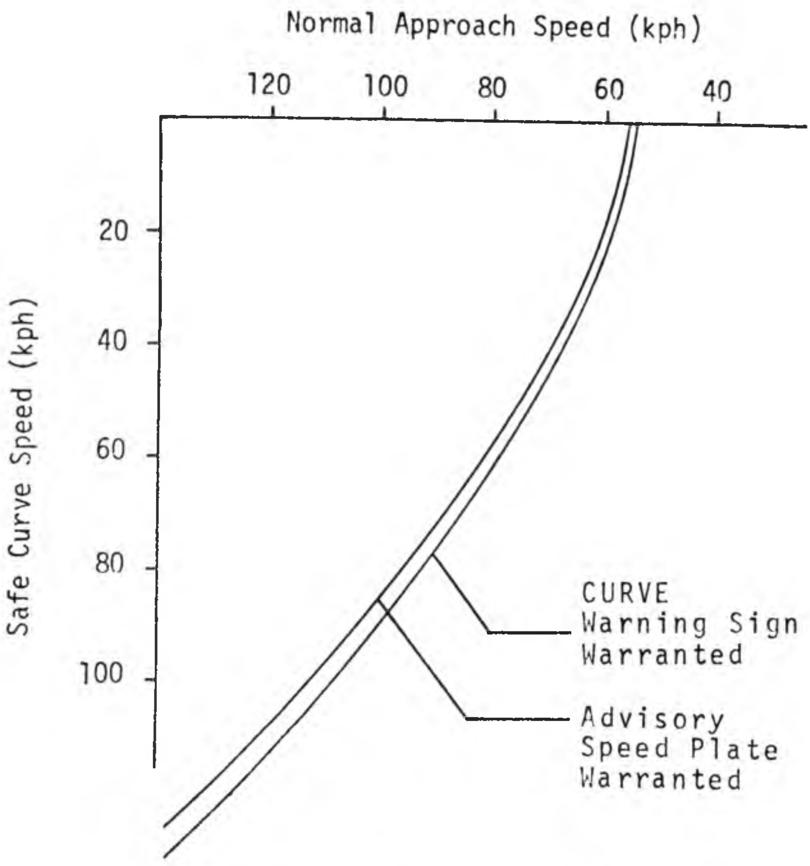


**Fig. 3. Required Deceleration Distances on Horizontal Curves**
**Note: 1 kph = .625 mph, 1 meter = 3.3 ft**

respect to the MUTCD, requires the use of standard no-passing-zone stripes on all such sections. As this practice may be unnecessarily expensive, an evaluation of the need for such a practice is necessary. The probability of conflict technique was again employed for this determination.

*Analysis*

For analysis purposes, it was assumed that all passing maneuvers were undertaken without regard for oncoming vehicles; i.e., as soon as a driver overtakes a slower vehicle, he pulls out to pass. This assumption produces unrealistic results which will be adjusted later.

The basic situation for development of probability of conflict is as follows:

> A driver in vehicle A, traveling 80 kph (50 mph), overtakes vehicle B traveling 64 kph (40 mph). Without regard for safe passing sight distance, the driver in vehicle A pulls into the opposing traffic lane to pass vehicle B. Before vehicle A can return to the right lane, vehicle C, traveling in the opposite direction, comes into conflict with vehicle A.

The necessary determination in this evaluation is the probability of the above situation occurring.

To begin with, the probability of vehicles A and B being at the above passing situation is the probability of simultaneous arrival (within a $\Delta t$ = two seconds) of two or more vehicles, given by:

$$P(x) = 1 - [P(O) + P(1)]$$

Based on the maximum low-volume rural road ADT of 400 (200 vpd in each direction), the probability of such an occurrence in any two-second interval is $4 \times 10^{-5}$ Over an entire day, the expected number of potential passing situations is 0.864.

Assuming that the following vehicle passes at his constant speed of 80 kph (50 mph), the length of time that vehicle A is encroaching on the opposing lane is determined as follows:

$$t = \frac{d}{.277v}$$

where, d = distance traveled in left lane (meters)
v = average speed (kph)
t = time left lane occupied

For an assumed speed of 80 kph (50 mph), the duration of encroachment on the opposing lanes is approximately 11 seconds. Therefore, if an opposing vehicle arrives during that 11-second interval, there will be a conflict. The probability of such an arrival [P(A)] in the opposing lane is 0.04965.

The probability of the passing maneuver occurring during the 11-second critical interval is:

$$P(P) \frac{11}{2} \times .00004 = .00022$$

The probability of both events occurring, thus causing a conflict, is the product of the respective probabilities:

$$
\begin{aligned}
P(C) &= P(P) \times P(A) \\
&= (.00022)\ (.04965) \\
P(C) &= 1.09 \times 10^{-5}
\end{aligned}
$$

Over the course of a year, the expected number of conflicts would be 15.6, or about one conflict every three weeks. However, this figure is based on total disregard for passing sight distance.

Assuming that there was about 30 percent passing sight distance on our example roadway, and that the ordinary prudent driver would take advantage of this visibility, the expected number of conflicts per year is reduced by 30 percent to about eleven.

Although this number may seem a bit high to be tolerable, it must be remembered that it applies to the worst case—400 ADT and total disregard for safety on sections on inadequate passing sight distance by all drivers. As it is probable that a majority of drivers would not attempt a passing maneuver without at least marginal sight distance, the actual number of conflicts is more likely to be two or three per year.

Yet this figure is applicable only for 400 ADT facilities. The average facility examined (about 150 ADT) would produce, over the long run, only about one conflict every three or four years.

This analysis indicates that there may be inefficiency of striping no-passing zones on low-volume rural roads as per MUTCD requirements. Such a practice *might* prevent a conflict every few years, and there is no reason to believe that every conflict will result in an accident. It is conceivable that a paint stripe would not prevent any accidents throughout the entire life of the paint.

### *Guidelines*

Although the probability of conflict in a passing maneuver has been shown to be minute, the elimination of all signs and markings relative to passing does entail some risk. Yet the degree of risk involved does not appear to justify the expense of standard MUTCD striping. The following alternatives are offered in lieu of MUTCD striping.

A PASSING HAZARDOUS warning sign should be used to indicate extended sections of inadequate extended sections of inadequate passing sight distance on all unmarked paved roadways and all unpaved roadways. Such signs should have attached a supplementary plate bearing the legend NEXT *XX* MILES, indicating the length of the section. Subsequent PASSING HAZARDOUS signs and supplementary plates should be erected beyond the intersections with paved roadways. The mileages on these subsequent supplementary plates should indicate the number of miles remaining in the section from that point.

If centerline definition is desired on paved roadways with insufficient passing sight distance, a double narrow line may be used in lieu of the PASSING HAZARDOUS signs. The double narrow line consists of two 1½-inch yellow lines separated by a 1-inch space. This line should be used only for extended sections of insufficient passing sight distance; intermittent sections of restricted sight distance within which striping is deemed necessary should be striped as per present MUTCD guidelines. As vehicle wheel paths on roadways less than 20 ft wide tend to overlap the centerline and obliterate painted pavement markings, such roadways should not be striped.

## SUMMARY

The results of this research indicate that considerable benefit can be derived from a reevaluation of the needs for signs and markings on low-volume rural roads. These benefits include not only obvious monetary savings from reduced levels of signing and marking, but also considerable savings in time and frustration on the part of the engineer responsible for the operation of these roadways. Guidelines presented herein were developed solely for the rural context, and are thus more readily applicable to that environment than the guidelines offered in the MUTCD. Although the recommendations presented by no means cover all control devices or all situations, they do provide guidance in three most crucial areas—intersections, horizontal curves, and no-passing zones.

### *Intersections*

Low-volume rural intersection control can be efficiently achieved through guidelines based on an economic analysis. Primary variables governing the application of regulatory/warning devices are approach speed, ADT, and sight distance. Below 200 vpd combined entering volume, STOP control is inefficient and should not be used except in rare cases. CROSS ROAD signs are advocated for use in lieu of STOP signs at certain locations described in the guidelines.

### *Horizontal Curves*

Existing signing practices produce more curve warning signs than are necessary. The guidelines presented describe a more efficient and pragmatic technique for signing of horizontal curves. It was shown that this reduced level of signing did not adversely affect safety as drivers tended to be more cautious on unsigned curves.

### *No-Passing Zones*

Guidelines were developed that are more efficient than existing standards for traffic control in sections of inadequate passing sight distance. Analyses showed that the potential for accidents in no-passing zones is virtually nil on these roadways. Recommendations contained herein would virtually eliminate standard striping of no-passing zones and replace that practice with 1) PASSING HAZARDOUS sign, or 2) a more economical double narrow line.

The authors found, in general, that standard practices for signing and marking of highways are inefficient and unsuited to the rural environment. The recommended guidelines should provide for a much more orderly, pragmatic, and efficient application of control devices on low-volume rural roads.

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## DISCLAIMER



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