Modeling Conflicts at Intersections Hidden by Vertical Curves

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In this paper, a model that estimates the relative hazard to passenger cars stopping to turn left at an intersection hidden by a vertical curve is described. Because of the limited sight distance, following cars might not be able to see the left-turning vehicle in time to stop on wet pavement. Monte Carlo methods are used to estimate the relative frequency of such hazardous incidents as a function of traffic volume and sight distance. These incidents are serious conflict situations in which an accident could not be avoided by braking alone. Left-turn gap acceptance and headway distributions in opposing traffic are used in the model to determine the random delay experienced by left-turning passenger cars. Other random variables addressed in the model are traffic speeds, headways of cars following the left-turning car, and the perception reaction time of following car drivers. The results indicate that the conflict rates increase rapidly with decreasing sight distance. Conflicts increase with hourly volume up to about 600 vph and then level off or decrease. Values range from 9 conflicts per 10,000 wet-pavement left turns at 150 vph at sites with sight distance based on the 1965 AASHO Bluebook to 486 conflicts per 10,000 wet-pavement left turns at 750 vph and substandard sight distance. Assuming that 2 percent of traffic turns left and that pavements are wet 15 percent of the time, annual totals range from 0.32 to 101 conflicts per year, depending on sight distance and daily volume. Conflict rates are substantially reduced by decreases in reaction time and speed, which might be produced by appropriate signing, and increases in pavement friction. In combination, these countermeasures are at least as effective in reducing conflicts at sites with substandard sight distance as bringing the sight distance up to standard would be.

The subject of this paper is a Monte Carlo simulation model developed to analyze the hazard to cars stopped to turn left at an intersection hidden by a vertical curve. AASHTO (originally AASHO) policy for the geometric design of vertical curves is intended to ensure adequate stopping sight distance to a small (6-in.-high) obstacle *(1)*. Larger objects such as pedestrians or other vehicles represent greater intrinsic hazards but are visible at much greater distances. These distances are less likely to be exceeded in emergency stops than the AASHTO design sight distance. Nevertheless, the same geometry that governs sight distance to the AASHTO target also determines sight distance to other obstacles. In a substandard vertical curve design, all sight distances are diminished, and under certain conditions even other vehicles may be seen too late for drivers to stop safely.

In this study, a simulation model is used to estimate the relative hazard arising from varying degrees of sight distance restrictions at an intersection. The model creates a scenario in which a passenger car stops to make a left turn at an intersection or a driveway hidden by a vertical curve on a two-lane highway. Because of the limited sight distance, following drivers might not be able to see the left-turning vehicle in time to stop, especially on wet pavement. The model estimates the number of such hazardous incidents per 10,000 wet-pavement left turns as a function of sight distance, traffic volume, and other traffic and site characteristics. The hazardous incidents counted by the model are not necessarily accidents; however, they do represent serious conflict situations in which an accident cannot be avoided by braking alone. The terms "conflict" and "hazardous incident" are used interchangeably throughout the paper to refer to such events.

For the sake of brevity, the word "model" is used here in referring to the computations described in this paper. "Figure-of-merit algorithm" would be a more apt term. The degree to which the logic and calculations that make up this algorithm represent reality is difficult to validate empirically. In fact, if one had the detailed accident data required for validation, there would be no need for the model. At present, an implicit figure of merit for a given vertical curve is obtained by comparing the AASHTO sight and stopping distances. The site is regarded as more or less safe as the sight distance exceeds the stopping distance to a greater or lesser degree. Clearly, many factors other than sight and stopping distance influence the degree of hazard at a given vertical curve site. The conflict algorithms described here take many of these into account. The conflict rate calculated by the model should thus be regarded as a more sophisticated and perhaps more realistic figure of merit than a simple comparison of sight and stopping distances, and one which may be more closely related to actual accident rates.

BACKGROUND DATA AND ASSUMPTIONS

The Left-Turn Scenario

The focus of the simulation is a northbound vehicle making a left turn from a two-lane rural road onto an intersecting road or driveway that is on, or just north of, a vertical curve. This vehicle is assumed to be a passenger car. The sight distance available to northbound vehicles is restricted by the vertical curve. If the left-turning car is delayed by southbound traffic and has to stop, it may be struck in the rear by a following vehicle, also assumed to be a passenger car. The traffic relationships and road geometry are shown in Figure 1.

In Figure 1, Following Car 1 is within sight of the left-turning car as it slows to make the turn at the intersection, and

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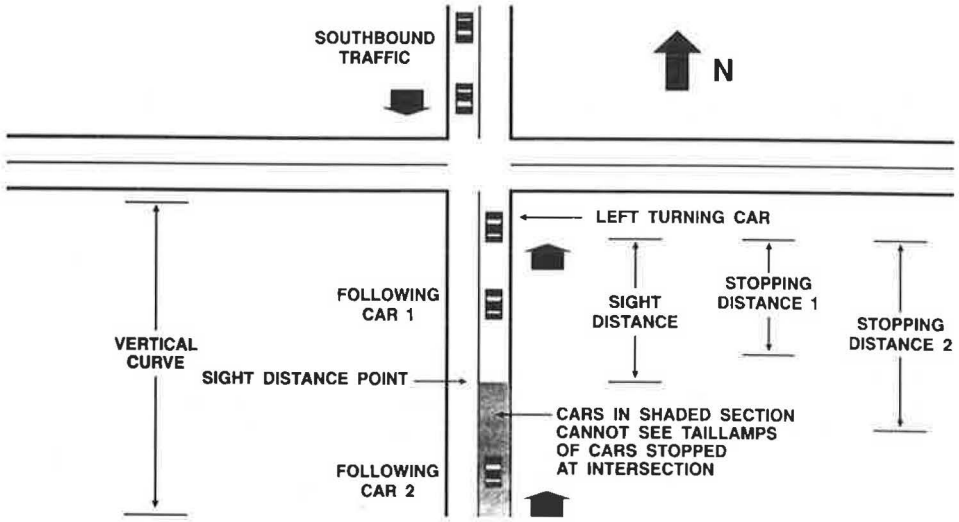

FIGURE 1 Plan view of vertical curve.

is therefore not at risk because of the sight distance restriction. The driver of Following Car 2, however, will not see the left-turning car until Car 2 reaches the sight distance limit point. If Car 2's stopping distance is less than the sight distance (e.g., Stopping Distance 1 on the figure) Car 2 will be able to stop in time. However, if the stopping distance is greater than the sight distance (Stopping Distance 2 in the figure) the following car will not be able to brake in time to stop behind the left-turning car.

Whether or not such an incident occurs depends on many factors: the headways and speeds of the following vehicles and the reaction times of their drivers, headways in opposing traffic, driver left-turn gap acceptance characteristics, pavement friction, vertical curve geometry, driver eye height, and the visually effective height of the left-turning car. In the model, some of these variables are given fixed values, some are fixed for a given site, and some vary randomly from vehicle to vehicle. The model uses a Monte Carlo procedure to deal with the random variables. The random variables, for example, vehicle speed, are represented by probability distributions. Each iteration of the model begins with a northbound vehicle making a left turn at the intersection. For each such iteration, the model randomly samples each of the distributions of random variables. The model uses these values to determine whether or not a following car will see a stopped left-turning car or a car queued behind the left-turning car in time to stop. If not, a serious conflict is assumed to occur. This determination is made for each iteration of the model and a running count of the number of such conflicts is accumulated.

The detailed steps the model goes through to make this determination are described in detail in a later section. The remainder of the present section provides background on the data sources, analyses, and assumptions on which the model is based.

Headway Distributions and Left Turn Delays

Headway distributions are used in the model for both northbound and southbound traffic. For northbound traffic, the headway distributions determine the spacing of the potentially striking vehicles following the left-turning car. The headway distributions were also used to represent the gaps in southbound traffic that determine the delay experienced by the left-turning car. The headway distribution used is the so-called "Hyperlang" distribution developed by Dawson *(2)*. Dawson's model was selected because (a) it is in an analytical form suitable for programming, (b) it was developed to represent headways on two-lane highways, and (c) it has been validated against actual headway distributions. The particular form of the model used here is the one parameterized to fit the headway distributions given in the 1965 Highway Capacity Manual *(3)*, Figure 3.29.

A separate Monte Carlo program was written to generate probability distributions of delays for the left-turning vehicle. These distributions give the probability that a northbound vehicle arriving at the intersection to make a left turn will experience a delay equal to or greater than a given duration. Ten delay distributions for each of 10 levels of traffic volume ranging from 150 to 1,050 vph were computed. Each of these distributions is based on a random sample of 5,000 cases.

The Monte Carlo routine used to generate these distributions makes use of two distributions in addition to the Hyperlang headway distribution. These were (a) left-turn gap acceptance probability as a function of gap size, and (b) probability distributions of next-arrival times at the intersection for southbound traffic at various volumes. The gap acceptance distribution was developed by fitting a log normal curve to empirical left-turn gap acceptance data reported by Betz and Bauman *(4)*. This distribution has a median of 6 sec and a standard deviation of 1.4 sec. A gap acceptance distribution, rather than a single critical value, was used because drivers who demand long gaps are most likely to experience long delays.

The next-arrival distribution is the cumulative distribution of next-arrival times, the time duration from the arrival of the left-turning car at the intersection to the arrival of the next opposing vehicle at the same point. This distribution was generated by weighting the relative frequency of each headway in the highway distribution by the headway itself. This procedure reflects the fact that the likelihood that a car at an arbitrary point on the road is within a gap of a given size is proportional to both the

relative frequency of gaps of that size and the gap size itself. Next-arrival distributions were generated for traffic volumes ranging from 150 to 1,050 vph.

The procedure for computing delay distributions from the gap acceptance and next-arrival distributions is described here and shown in Figure 2. The procedure is for one case. First, a critical gap is randomly chosen from the gap acceptance distribution. This is the threshold gap, the minimum gap in opposing traffic the driver will accept to make a left turn. Then a next-arrival time is randomly drawn. If the critical gap is less than the next-arrival time, the turn is made and the delay for the present case is zero. If the critical gap is greater than the next-arrival time, the turn is not made and the delay will be at least equal to the next-arrival time. In this case, the left-turning car comes to a stop at the intersection to wait for an acceptable gap in southbound traffic. A random headway is drawn from the Hyperlang distribution. If the critical gap is less than the headway, the turn is made and the delay is set equal to the next-arrival time. If the critical gap is equal to or greater than the headway, the turn is not made, the delay is increased by the headway, and a new random headway is drawn. This cycle is repeated until a headway is drawn that is equal to or greater than the critical gap. The total delay for the case is equal to the sum of the next-arrival time and the headways accumulated up to that point. This procedure is repeated until a sample of 5,000 cases is generated.

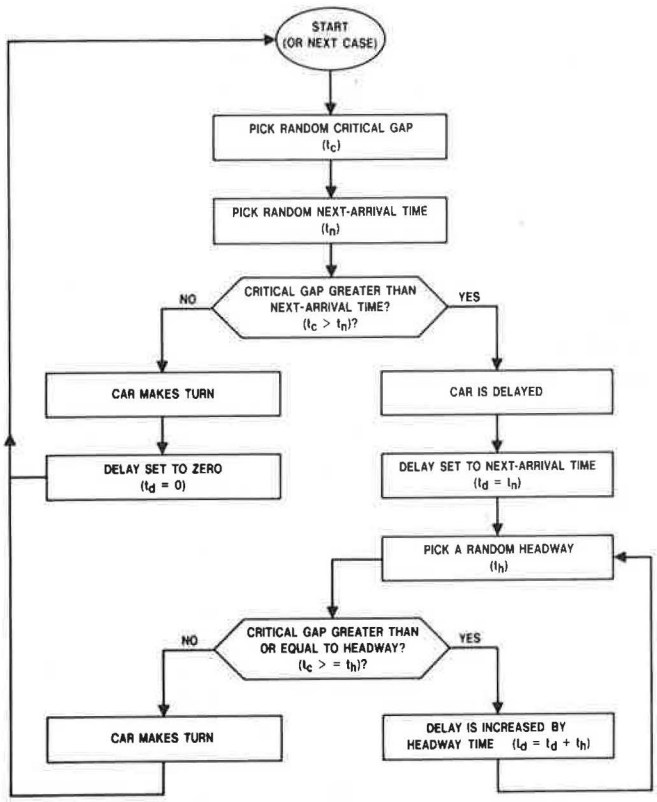

FIGURE 2 Creating a delay distribution—logical flow.

Computing Sight Distance on Vertical Curves

The left-turning or other northbound vehicle on the vertical curve is assumed to be visible to a following driver if its tail lamps are visible. During the day, this assures that more than one-half of the vertical extent of the lead car is unobscured by the crest. At night, the tail lamps themselves are clearly visible.

The expression used to compute sight distance is that given in the 1984 AASHTO Greenbook *(1)*. The expression has length of curve as the independent variable:

$$L = \frac{AS^2}{100[(2h_1)^{1/2} + (2h_2)^{1/2}]^2}$$

where

A = algebraic difference in the grades (percent),
S = sight distance (ft),
h_1 = eye height (ft),
h_2 = object height (ft), and
L = length of vertical curve (ft).

An existing hillcrest designed to conform to AASHTO guidelines would be based on the 1965 AASHO Bluebook *(5)* value for driver eye height rather than on the 1984 AASHTO Greenbook *(1)* value. Assuming the 1965 AASHO Bluebook values of 3.75 and 0.5 ft, respectively, for h_1 and h_2 on a vertical curve with $A = 6$, a curve length L of 1,545 ft is required to provide 600 ft of sight distance. The object of interest in the present case is the rear of a vehicle, in particular, the tail lamps. A typical value for tail lamp height in current vehicles is 2 ft. The 1984 AASHTO Greenbook design eye height for vertical curves is 3.5 ft. This is close to the minimum eye height in current passenger vehicles *(6)*. Using 3.5 ft to represent driver eye height in current vehicles and an object height of 2 ft to represent the tail lamps, that same 1,545-ft curve would provide 746 ft of sight distance, an increase of 24 percent. This increase produced by the new values of h_1 and h_2 is constant for all values of A and L.

Stopping Distance

The expression used is

$$d = 1.467vt + v^2/30f$$

where

d = stopping distance (ft),
v = speed (mph),
t = reaction time (sec), and
f = coefficient of friction between the tires and the road.

The basis for selecting given values of speed, pavement friction, and reaction time is discussed next.

Traffic Speed Characteristics

In the model, the speeds of the northbound vehicles following the left-turning car are important because they determine stopping distances. Speed is treated as a random variable in the

model. Speed is assumed to be normally distributed with a standard deviation of 7 mph. Olson et al. *(7)* report data showing standard deviations ranging from 4.7 to 14.9 mph. A typical value is 7 mph. Using a higher standard deviation in the present model would likely lead to somewhat greater conflict rates because higher speeds and hence longer stopping distances would occur more frequently.

Pavement Friction

The baseline pavement friction value used in the model is based on an *SN*40 value of 33. This low value can be taken as a reasonable worst case for skid resistance. Effective tire-pavement friction decreases with speed. According to Olson et al., the effective speed for determining a representative pavement friction value for a vehicle braking to a stop is 0.707 times the initial speed. Farber et al. *(8)* provide data relating maximum deceleration levels to different wet-pavement *SN*40 values. The general form of the relationship is

$$f = 0.69SN40 - k$$

where $k = 18.3 - 0.31\text{Ve}$, and Ve is effective braking speed (mph). Then, if V is the actual speed in feet per second,

$$k = 18.3 - (0.707)(0.31)V/1.467 = 18.3 - 0.15V, \text{ and}$$
$$f = (0.69SN40 - 0.15V + 18.3)/100$$

At 60 mph (88 ft/sec), for example, the value for f is 27.6. The values of f that this formula produces, assuming an *SN*40 value of 33, are close to the values used in the 1965 AASHO Bluebook *(5)* for computing stopping sight distances.

Reaction Time

Reaction time is treated as a random variable in the model. The distributions of reaction time are based on the research of Olson et al. *(7)*. Olson et al. conducted a study to measure the perception-reaction response time (referred to here simply as "reaction time") of drivers exposed to a 6-in.-high obstacle hidden by a vertical curve. The time was measured from the instant that the obstacle was first in view until the driver's foot contacted the brake pedal. The median reaction time was 1.1 sec; the 5th and 95th percentile reaction times were 0.85 and 1.60 sec, respectively. These data were based on a single surprise trial for each subject. Even though the subjects were not expecting the obstacle, the study was conducted in the presence of the experimenters, a factor that could produce heightened attention. Accordingly, Olson et al. recommended increasing the 95th percentile reaction time by 50 percent to 2.4 sec as a design reaction time to represent unalerted drivers.

To make use of these data in the present research, two log normal distributions were developed. The first was fitted to the original data; the second to the stretched data with the 2.4-sec 95th percentile reaction time. The parameters of the distributions were as follows:

	Alerted Driver (original distribution)	*Unalerted Driver (modified distribution)*
Median	1.10 sec	1.43 sec
5th percentile	0.85 sec	0.85 sec
95th percentile	1.60 sec	2.40 sec
Standard deviation (of log normal distribution)	0.228	0.318

The baseline distribution used in the model was the modified version with median of 1.43 sec and 95th percentile of 2.40 sec. The original distribution with median of 1.10 sec was used to represent the influence of signing or some other countermeasure that might increase driver attention.

Start-Up Delays

In the model, a queue may form behind a delayed left-turning car. The last car in the queue rather than the left-turning car itself becomes the vehicle at risk. The total delay experienced by this car is equal to the left-turn delay plus the start-up time, the time for the *N*th vehicle in the queue to begin moving after the left turn commences. Calculation of the start-up delay is given by a formula provided by C. Messer (personal communication). The original expression is

$$\text{Delay} = 2N + k$$

where delay is given in seconds, N is the number of vehicles in the queue, and $k = 2$ sec. The value of 2 sec for k is for queues in which the lead vehicle is proceeding straight. For the present application, in which the lead vehicle is turning left, k was set equal to 3 sec. This value was based on the observation of the time required for an unimpeded left-turning vehicle to clear its lane and the following car to commence acceleration.

THE LEFT-TURN SCENARIO MODEL

The left-turn scenario model and Monte Carlo procedure are described in this section. The sequence of steps and the logical relationship between them are shown in the flowchart in Figure 3. Each iteration of the model begins with a car approaching the intersection in the shadow of the vertical curve to make a left turn.

Determining Left-Turn Delay

The first step is to determine whether or not the left-turning car is delayed by southbound traffic and, if so, for how long. To make this determination, a random delay is drawn from a previously computed probability distribution of left-turn delays. If the delay is zero, it is assumed that the left-turning vehicle completes its turn safely. The no-conflict counter is incremented and a new case is drawn.

The left-turning car might be struck by a following vehicle as it slows approaching the intersection. In order for this accident to happen, it would be necessary for the following car to be well within the available sight distance. Such an accident

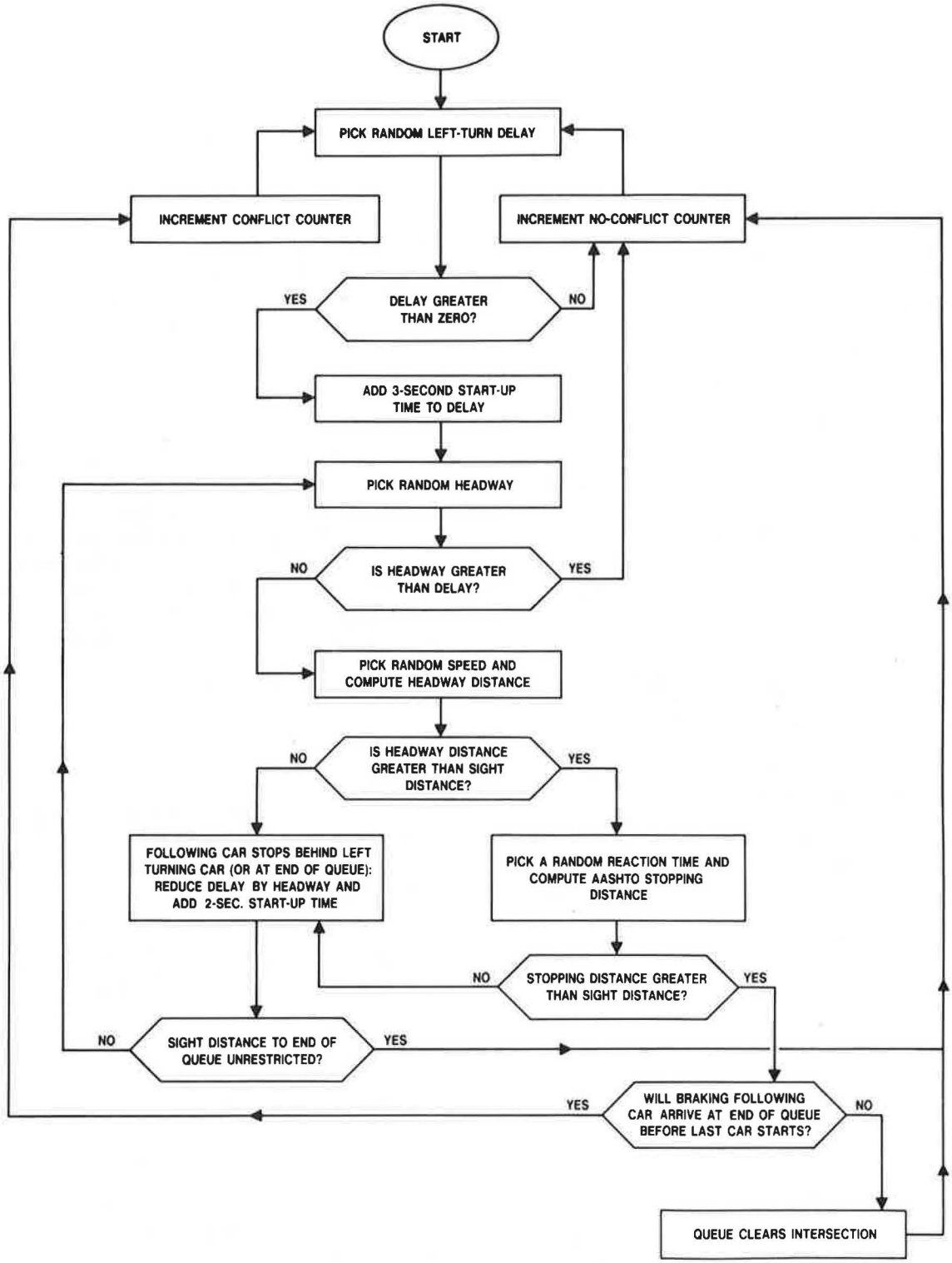

FIGURE 3 Left-turn scenario—logical flow.

would be due to inattention on the part of the following car driver, not to the vertical curve, and is therefore not considered by the model.

If the left-turning car is delayed, it is assumed to come to a full stop at the intersection. Three seconds are added to the delay at this point to account for start-up time. This delay can be regarded as the left-turning car's exposure.

Determining Following Car Headway

The model next draws a random headway time from a distribution of headways. This is the headway between the left-turning car and the next following northbound car. The headway is measured at the sight distance limit point, the point on the vertical curve at which the driver of a passenger car cresting the

hill can just see the tail lights, assumed to be 24 in. off the ground, of a car ahead. This sight distance is about 24 percent longer than the AASHTO sight distance.

Comparing Headway to Delay

If the headway is greater than the delay experienced by the left-turning car, the left-turning car will clear the northbound lane before the following car arrives, and no incident occurs. Again the model increments the no-conflict counter and loops back to take the next sample.

If the headway is less than the delay, the following car will arrive at the intersection before the left-turning car can depart. What happens then depends on a number of factors.

Comparing Following Distance to Sight Distance

A random speed is drawn for the following car, and the following distance (speed × headway) is computed. If this distance is less than the sight distance, it means that the left-turning car was visible to the following car as it slowed to make the turn. In this case, no conflict attributable to the sight distance restriction occurs, and the following car comes to a stop behind the left-turning car. The following car now becomes the vulnerable vehicle. The following car's exposure, that is, how long it is delayed by the left-turning car, is determined by subtracting the original headway from the delay and adding 2 sec for the following car's start-up time. At this point, the next following car's headway is drawn and these computations are repeated. Following-car headways are drawn and the queue and delay are adjusted until one of the following occurs:

1. A headway is drawn that is greater than the remaining delay, in which case the queue clears and there is no conflict. The no-conflict count is incremented and the model loops back to take the next case.
2. The queue of following cars stopped behind the left-turning car becomes so long that it exceeds the length of the vertical curve and the sight distance to the last car in the queue becomes unrestricted. In this event, the hazard disappears. The no-conflict counter is incremented and the model loops back to take the next case.
3. A headway is drawn that is less than the delay, and the randomly drawn speed is such that the following distance is greater than the sight distance.

In this latter case, the following car driver will not see the left-turning car or the last car in the queue until it arrives at the sight distance limit point. Nor will the last car have departed before the following car arrives at the end of the queue, if the following car doesn't brake. The next step is to determine whether or not the following car can brake to a stop within the available sight distance.

Determining Stopping Distance

A random perception-reaction time is drawn for the following-car driver and used to compute a total stopping distance. If this distance is less than the sight distance, the following car joins the queue and the next following vehicle is examined. If the stopping distance is greater than the sight distance, there will likely be a conflict. However, because the following car is braking, the time it takes to arrive at the end of the queue is somewhat greater than the headway time, and thus the queue might still clear before it arrives. Accordingly, the time required for the braking car to travel from the sight distance limit point to the rear of the queue is computed. If this time exceeds the remaining delay, there is no conflict, in which case the no-conflict counter is incremented and the next case is drawn. However, if this duration is less than the delay, the braking following car will arrive at the queue with speed greater than zero and a hazardous incident will occur. The conflict counter is incremented and a new case is drawn.

RUNNING THE MODEL

The model was run under a number of different conditions. On each run, a sample of at least 50,000 left-turning vehicles was used. The output of each run is expressed as the number of hazardous incidents per 10,000 left-turning cars. In preliminary runs, it was found that the hazardous incident rate on dry pavement was a small fraction of the wet-pavement rate. Accordingly, all subsequent exercises were run under wet conditions. The parameters varied were as follows:

- Traffic volume: 150 to 900 vph
- Traffic speeds:
  - Mean: 45 or 60 mph
  - Standard deviation: 7 mph
- Sight distance:
  - 275 and 350 ft (mean traffic speed, 45 mph)
  - 475, 550, and 600 ft (mean traffic speed, 60 mph)
- Pavement friction: $SN40$ = 33 or 40
- Reaction time:
  - Base reaction time for unalerted driver:
    - Median = 1.43 sec
    - Standard deviation (of log normal distribution) = 0.318
  - Adjusted reaction time for alerted driver:
    - Median = 1.10 sec
    - Standard deviation (of log normal distribution) = 0.228

Traffic Volume and Sight Distance

A series of runs was made to determine how the predicted conflict rate varies with traffic speed and the design sight distance. Mean traffic speeds of 45 and 60 mph were investigated. It is normal design practice to use at least the 85th percentile speed of traffic as the design speed. This is about one standard deviation (i.e., 5 to 10 mph) greater than the mean speed. A road for which average traffic speeds of 60 mph were anticipated would normally be built for a design speed of 70 mph. Accordingly, the baseline 60-mph run was made with a sight distance of 600 ft, the 1965 AASHO Bluebook *(5)* value for a 70-mph design speed. Additional runs were made with sight distances of 550 and 475 ft, 1965 AASHO Bluebook practice for design speeds of 65 and 60 mph, respectively. At 45 mph, runs were made at sight distances of 350 and 275 ft, corresponding to 1965 AASHO Bluebook design speeds of

50 and 40 mph, respectively. Sight distance and traffic speed combinations were selected with regard to the 1965 AASHO Bluebook, rather than the 1984 AASHTO Greenbook *(1)* design values, because the older standard was in effect when existing roads were built. These 1965 AASHO Bluebook sight distances assumed a 3.75-ft eye height and a 6-in.-high target. The current value for eye height, as given in the 1984 AASHTO Greenbook, is 3.5 ft. The sight distances to the 2-ft-high tail lamps from a 3.5-ft eye height are about 24 percent longer than the 1965 AASHO Bluebook sight distances to the 6-in.-high obstacle. The value of *A*, the algebraic sum of the slopes of the two tangent sections, was set equal to 6 for all runs.

Numerical results are given in Table 1 and shown graphically in Figure 4. According to these results, the conflict rate increases with traffic volume up to 600 to 750 vph, then levels off or decreases. There are two processes operating in opposite directions that could account for this effect. As the volume increases, the delay, and hence the exposure, experienced by the left-turning car or the cars queued behind it increases. At the same time, however, the likelihood that the following driver will have the left-turning car in view as it slows to turn increases with increasing volume. Also, at the heavier volumes, long queues can develop. At some point, as the rear of the queue approaches the tangent point of the vertical curve, the visibility to the last car in the queue becomes unrestricted and the hazard dissipates. This queuing effect probably accounts for the sharp downturn in the 275-ft results. The effects of the traffic volume on conflict rates at hourly flows greater than 600 vph are probably not an important practical issue in any case, because sustained flows exceeding this level are rare on two-lane rural roads.

TABLE 1 NUMBER OF CONFLICTS PER 10,000 WET-PAVEMENT LEFT TURNS FOR DIFFERENT TRAFFIC VOLUMES AT VARIOUS COMBINATIONS OF TRAFFIC SPEED AND AASHTO SIGHT DISTANCE

	AASHTO SIGHT DISTANCE/CURVE LENGTH (FEET)				
	600	550	475	350	275
	1525	1298	968	526	324
	CORRESPONDING 'BLUEBOOK' DESIGN SPEED (MPH)				
	70	65	60	50	40
TRAFFIC VOLUME (VPH)	MEAN TRAFFIC SPEED (MPH)				
	60	60	60	45	45
150	9	22	59	26	99
300	17	46	128	47	212
450	27	61	194	60	314
600	30	66	239	87	413
750	26	77	245	96	486
900	20	65	266	86	393

The conflict rates increase sharply as the sight distance decreases for a given mean traffic speed. At a mean traffic speed of 60 mph, the conflict rate is 7 to 10 times higher at a sight distance of 475 ft than at 600 ft. Similarly, at 45 mph, the rate is four to five times higher with 275 ft of sight distance than with 350 ft. It is instructive to consider these results in light of the relationship between the mean traffic speed and the assumed design speed. The hazardous incident rate at a 45-mph mean speed and 350 ft of sight distance is comparable to the rate at 60 mph and 550 ft, especially at the lower traffic volumes. In both cases, the sight distance corresponds to a 1965 AASHO Bluebook *(5)* design speed that is 5 mph higher than the mean speed. At the 275-ft sight distance, the design speed is 5 mph less than the mean speed. The resulting hazardous incident rate is about 30 percent more than in the 60-mph, 475-ft case in which the mean speed and design speed are the same. It is interesting to note that the 600-ft sight distance case, which conforms fully to 1965 AASHO Bluebook practice, has low conflict rates.

Effects of Traffic Speed, Reaction Time, and Pavement Friction

A series of runs were made to investigate the effects of traffic speed, driver reaction time, and pavement friction levels on the conflict rate. The base and modified reaction time distributions discussed earlier were used to represent alerted and unalerted drivers, respectively. The results of these runs are presented in Table 2 and shown in Figure 5.

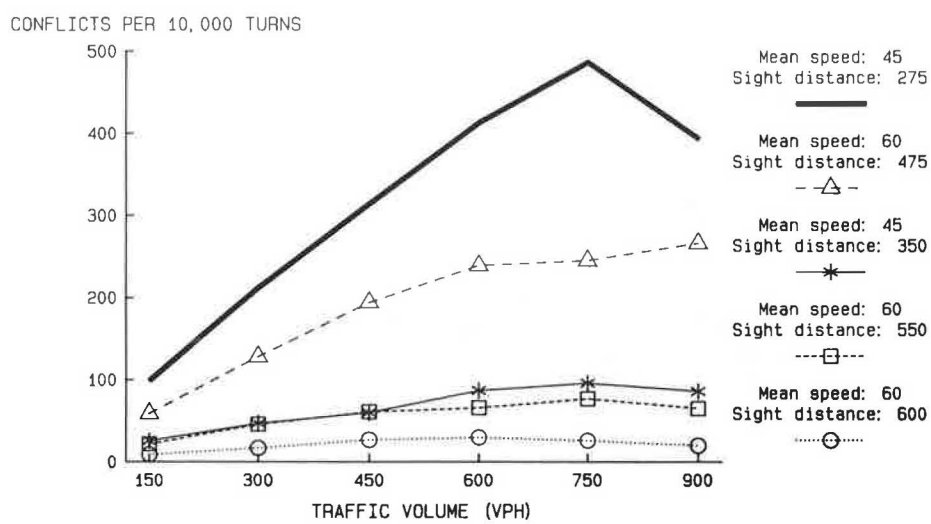

FIGURE 4 Number of conflicts per 10,000 left turns under different traffic and geometric conditions.

As might be expected, conflict rates were lower with lower speeds, higher pavement friction levels, and quicker reaction times. What is striking is the effect of the changes in combination. At the lower speed and reaction time and higher pavement friction, the rate is reduced by a factor of 18 relative to the base rate.

Projecting Annual Conflict Counts

The results in Tables 1 and 2 can be projected to provide estimates of the annual number of conflicts arising out of the left-turn scenario. To make these projections, it is necessary to weight the conflict rates associated with the various hourly volumes by the percentage of total traffic occurring at each volume. A typical rural road volume distribution curve from the 1965 Highway Capacity Manual *(3)* was used. The 1965 edition of the Highway Capacity Manual was used as the source for this information rather than the more recent edition *(9)* because the volume distribution curve in the earlier version is apparently based on more data and therefore seems more representative. This curve (*3*, Figure 3.6) shows the weekday hourly volumes over a 24-hr period on two-lane rural roads, expressed as a percentage of the AADT. It is also necessary to specify the fraction of left-turning vehicles and the fraction of time the pavement is wet. Values of 2 and 15 percent were used for left turns and wet pavements, respectively.

TABLE 2 NUMBER OF CONFLICTS PER 10,000 WET-PAVEMENT LEFT TURNS FOR DIFFERENT COMBINATIONS OF SPEED, REACTION TIME, AND PAVEMENT FRICTION

Traffic Volume - 450 VPH
AASHTO Sight Distance - 475 feet

Mean Traffic Speed	Reaction Time (Sec) Mean	Reaction Time (Sec) Log S.D.	Pavement Friction (SN-40)	Conflicts per 10,000 Wet Surface Left Turns
60	1.43	.317	33	163
55			33	60
60	(Base Distribution		40	84
55	Unalerted Driver)		40	23
60	1.1	.188	33	121
55			33	29
60	(Adjusted Distribution		40	34
55	Alerted Driver)		40	9

TABLE 3 PROJECTED ANNUAL NUMBER OF CONFLICTS BY AADT AND SIGHT DISTANCE FOR 2 PERCENT LEFT TURNS AND 15 PERCENT WET PAVEMENTS

Mean Speed - 60 SN-40 - 33
Base Reaction Time Distribution

AASHTO Sight Distance	AADT 1000	AADT 2000	AADT 5000
475	2.2	8.8	55.5
550	0.79	3.2	19.6
600	0.32	1.3	7.9
Mean Speed - 45			
275	4.0	16.2	101
350	0.80	3.2	20.1

TABLE 4 PROJECTED ANNUAL NUMBER OF CONFLICTS BY AADT FOR DIFFERENT TRAFFIC SPEEDS, PAVEMENT FRICTION LEVELS, AND REACTION TIMES FOR 2 PERCENT LEFT TURNS AND 15 PERCENT WET PAVEMENTS

AASHTO Sight Distance: 475 feet

Mean Speed	SN-40	Reaction Time	AADT 1000	AADT 2000	AADT 5000
60	33	Base	2.2	8.8	55.5
60	40	Distribution	1.1	4.5	28.1
55	33	(Unalerted	0.80	3.2	20.0
55	40	Driver)	0.32	1.3	8.1
60	33	Adjusted	1.6	6.5	40.6
60	40	Distribution	0.55	2.2	13.8
55	33	(Alerted	0.38	1.5	9.4
55	40	Driver)	0.12	0.48	2.8

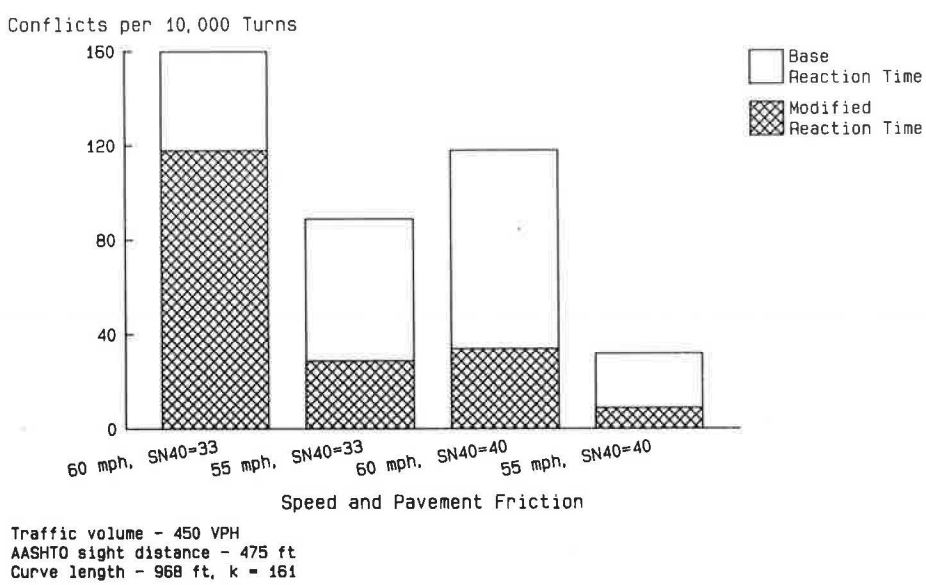

FIGURE 5 Number of conflicts per 10,000 left turns for different mean speeds, reaction times, and pavement frictions.

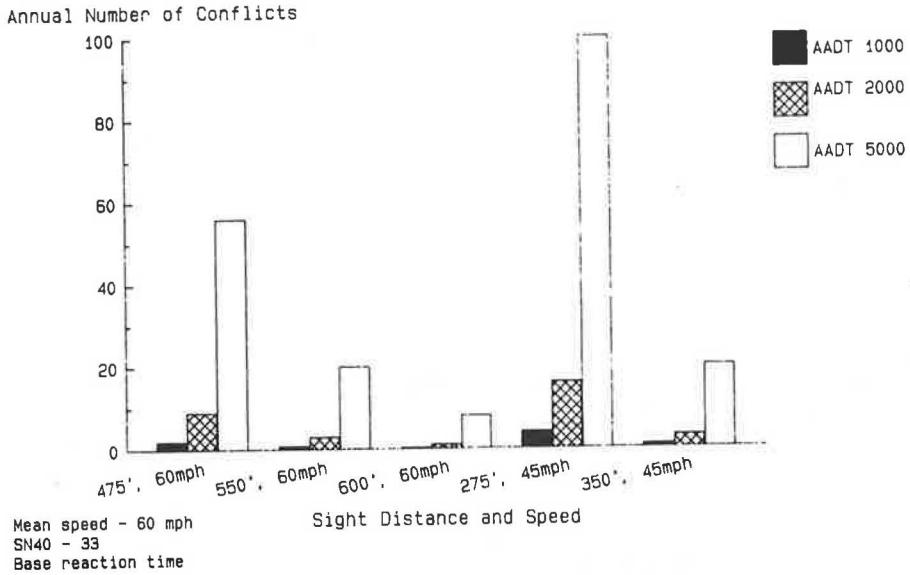

FIGURE 6 Projected annual number of conflicts by AADT and sight distance for 2 percent left turns and 15 percent wet pavements.

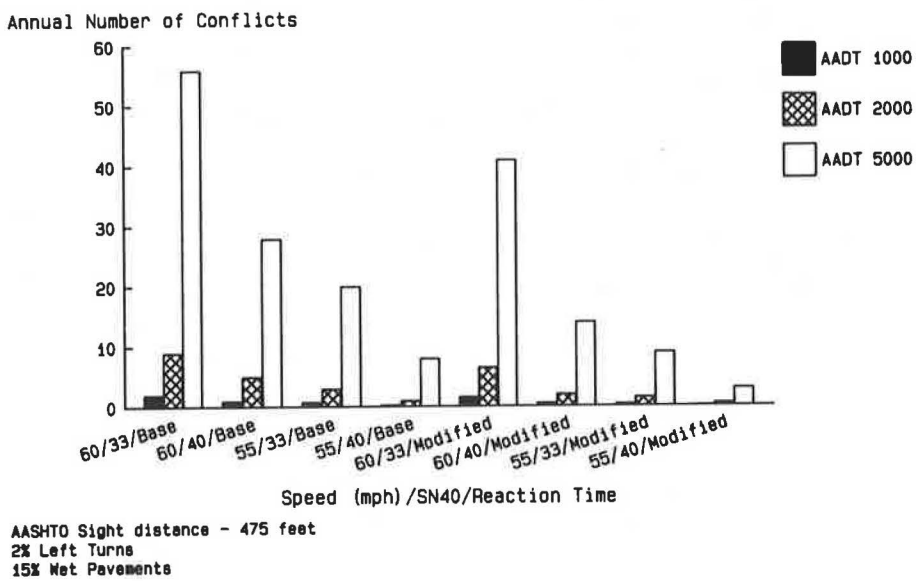

FIGURE 7 Influence of speed, pavement friction, and reaction time on projected annual conflict rates for different AADT values.

These estimates are given in Tables 3 and 4 for various site conditions and AADT levels. The same data are shown graphically in Figures 6 and 7. The tables and figures indicate that the projected annual incident rate increases rapidly with daily traffic volume. In fact, the number of conflicts increases with the square of the AADT. This is because the conflict rate and the annual exposure are both proportional to the traffic volume. In the case of the conflict rate, the proportionality is approximate and holds only up to 600 vph. Nevertheless, a relatively small proportion of total traffic flow occurs at higher volumes on two-lane roads, so the approximation is reasonable. Generally, the number of incidents is low, except for combinations of high AADT and substantially substandard sight distances. Even in this case, however, the results in Table 4 suggest the possibility of effective countermeasures. Increasing pavement friction and reducing speeds and reaction times lower the number of conflicts. These effects are specially strong in combination.

These resulting projections are specific to sites with these characteristics. However, because the conflict counts are directly proportional to the left-turn and wet-pavement rates, the projections in Tables 3 and 4 can be used to estimate the conflict counts for different left-turn and wet-pavement rates. For example, to estimate the projected number of conflicts for a site with a 7 percent left-turn rate, simply multiply the tabled count by 7/2.

The numbers in Tables 3 and 4 reflect the specific daily volume distribution cited previously. Although this distribution

may be typical of rural roads, there is certainly no basis for assuming that it is universally representative. Nevertheless, it would probably take a considerable difference in the volume distribution to change the outcome substantially. In any case, it would have no effect on the sensitivity of the counts to the roadway, traffic, and driver parameters.

DISCUSSION

Predicting Conflicts

As noted earlier, the hazardous incidents predicted by the model are not necessarily accidents; however, they do represent serious conflict situations in which an accident cannot be avoided by braking alone. The fact that the left-turning vehicle is delayed by opposing traffic means that the following car probably cannot safely swerve into the left lane to avoid a rear-end collision. Whether or not an accident can be avoided depends on many factors, including the skills of the driver, the geometry of the particular site, the presence of parked vehicles or other obstructions on the right shoulder, and the behavior of other traffic.

These imponderables make it difficult to do more than speculate as to the actual relationship between the incidents predicted by the model and actual accidents. It does seem likely, however, that the accident rate is sensitive to the same factors and to roughly the same degree as the incident rate, that is, factors having a large effect on the conflict rate would presumably have a commensurate effect on the accidents that arise from the conflict situation. The model is conservative in the sense that when it was necessary to make assumptions, the reasonable worst case assumption was made. The result is that the number of incidents is probably overestimated and can therefore be regarded as an upper bound on the number of accidents.

Countermeasures

Increases in pavement friction and decreases in speeds and reaction times, especially in combination, have a dramatic effect on the conflict rate predicted by the model. Increases in pavement friction are limited largely by cost considerations. However, the situation is more uncertain with regard to reducing speeds or reaction times. Two recent studies indicated that various signs designed to warn drivers of limited sight distance on vertical curves had little measurable effect on traffic. Christian et al. *(10)* observed the response of traffic to warning signs with the message "LIMITED SIGHT DISTANCE." The signs had no reliable effect on speeds. Freedman et al. *(11)* studied a number of different graphic and textual variations on the basic limited sight distance message that were designed to better convey the meaning. None of these had any measurable effect on traffic speeds or the response times of vehicles in traffic to a car parked on the side of the road in the visual shadow of the vertical curve. However, other studies have shown that warning signs with a strong attention-getting signal can reduce traffic speeds and increase drivers' awareness of the local hazard *(12–14)*. Also, Lyles *(15)* found that a sequence of regulatory and warning signs in advance of an intersection hidden by a vertical curve reduced traffic speeds by as much as 5 mph. Interviews with motorists passing through the site revealed that the signs also increased awareness of potentially conflicting traffic on the intersecting road. The present research indicates that the conflict rates arising out of the left-turn scenario are significantly reduced by small changes in these parameters.

Other Scenarios

Other conflict scenarios arising out of restrictions to sight distance on vertical curves were considered for analysis. These were (a) conflicts with vehicles crossing the north-south roadway from the hidden intersection, and (b) pedestrians crossing or walking in the roadway in the visual shadow of the vertical curve. Preliminary analysis suggests that the sight distance restriction creates no significant additional hazard in these situations.

Crossing Vehicles

In the case of crossing vehicles, a passenger car that enters the intersection just as a northbound vehicle appears at the crest of the hill will easily clear the intersection before the northbound vehicle arrives, even if the northbound vehicle doesn't brake. This is true, for example, for a northbound vehicle traveling at 75 mph on a vertical curve designed in conformance with 1965 AASHO Bluebook *(5)* guidelines for 60 mph, that is, with 600 ft of sight distance. A crossing vehicle accelerating at only 0.1 *g* would have traveled 40 ft, easily clearing the intersection, in the 5.45 sec it would take the northbound vehicle to cover the 600 ft.

Heavy trucks would take much longer than a passenger car to cross the intersection. However, the drivers of both the northbound vehicle and the crossing truck would each be able to see the other vehicle at far greater distances than the 1965 AASHO Bluebook sight distance. For example, on a hillcrest with 600 ft of 1965 AASHO sight distance, a truck driver with an 80-in. eye height would be able to see the headlights of a passenger car at the hillcrest at about 900 ft. The approaching passenger car driver would be able to see the truck at the height of its driver's eyes at 1,020 ft. This ability would give a northbound driver time to stop from 75 mph after a 3-sec perception-reaction time on a surface with an effective coefficient of friction of 0.27. This combination of speed and reaction time would occur roughly once per 10,000 vehicles. The probability that such a vehicle would appear at the hillcrest just as a heavy truck begins to cross the intersection on wet pavement is remote.

Pedestrians

A similar situation holds for pedestrians. Assuming that a person is visible when half of his or her body height is in view, a 5-ft-tall pedestrian would be visible to a passenger car cresting the hill at about 800 ft. The pedestrian would be able to see the passenger car headlamps at about the same distance. A car traveling at 75 mph would take more than 7 sec to cover the distance, even without braking. This is ample time for both pedestrian and driver to get out of the way. Pedestrian speed

while crossing at intersections is about 4.5 ft/sec. Even at 3 ft/sec, there would be more than enough time for the pedestrian to cross a 12-ft lane before the approaching vehicle arrived at the intersection. The argument that the pedestrian might not happen to notice or detect the approaching car for some time after it comes within view is not germane to the present discussion because such a problem is not caused by the vertical curve.

Sight Distance and Large Trucks

Only passenger cars were considered in the present application. Large trucks generally have longer braking distances than do passenger cars. However, truck drivers can see much farther on vertical curves than passenger car drivers because they sit so much higher. These two factors come close to balancing out so that there is probably little, if any, additional hazard when the following vehicle is a truck (*1*, Tables 8 and 9). If the left-turning vehicle is a truck, it will be visible to following vehicles at much longer distances than will a passenger car. At the same time, however, longer turning delays can be expected for trucks. Whether or not these factors would balance is not known. Without information on left-turn gap acceptance and clearance times for trucks, it is not possible to make a definitive model run to address the question. It may not be a significant issue, in any case, because large trucks make up a relatively small proportion of two-lane rural road traffic.

CONCLUSIONS

1. The conflict rates predicted by the model increase rapidly with decreasing sight distance and increasing hourly volume. Values range from 8 conflicts per 10,000 wet-pavement left turns at 150 vph at sites with sight distance based in the 1965 AASHO Bluebook to more than 500 conflicts per 10,000 wet-pavement left turns at 900 vph and substandard sight distance.
2. The conflict rates associated with large hourly volumes (more than 600 vph) contribute negligibly to the annual total because these volumes rarely occur.
3. The annual conflict rate increases with the square of the AADT. The rate is negligible at sites with moderate AADT and AASHTO sight distance. Annual totals assuming that 2 percent of traffic turns left and that pavements are wet 15 percent of the time range from 0.32 conflicts per year for a site with AASHTO sight distance and an AADT of 1,000, to more than 100 conflicts on a site with an AADT of 5,000 and seriously substandard sight distance.
4. Effective countermeasures are available for reducing the hazard at sites with substandard sight distance. Conflict rates are substantially reduced by decreases in reaction time and speed, such as might be produced by appropriate signing and increases in pavement friction. In combination, these countermeasures are at least as effective in reducing conflicts at sites with substandard sight distance as bringing the sight distance up to a standard would be.

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