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Design and Operational Effects of Geometrics

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Contents

Traffic Operations Study of the Turning Lanes on Uncontrolled Approaches of Rural Intersections

PATRICK T. MCCOY AND WELDON J. HOPPE

A time-lapse film study of the traffic operations on 14 intersection approaches on rural two-lane highways in Nebraska was conducted. The purpose of the study was to evaluate the safety effects of turning lanes on the uncontrolled approaches of intersections on rural two-lane highways. The turning lanes evaluated were left-turn, right-turn, and fly-by lanes. Traffic operations on the approaches with turning lanes were compared with those on similar approaches without turning lanes to determine the safety effects of the turning lanes. The measures of safety effectiveness used in the study were *(a)* standard deviation of mean approach speed, *(b)* traffic conflict rate, *(c)* frequency of abnormal turning maneuvers, and *(d)* improper lane utilization. Lower values of these measures were assumed to be indicative of safer traffic operations. The results of the study indicated that the provision of turning lanes on uncontrolled approaches of intersections on rural two-lane highways improved the safety of traffic operations on these approaches, especially those without paved shoulders. It was also apparent from the results of the study that consideration must be given to the adequate design of these lanes, particularly left-turn and fly-by lanes, in order to eliminate their improper use and encroachments by turning vehicles on adjacent through lanes, which negate the safety benefits provided by such lanes.

Turning lanes are provided on uncontrolled approaches of intersections on rural two-lane highways to remove the deceleration and storage of turning vehicles from the through traffic lanes and thereby enable through vehicles to pass by without conflict and delay. Thus, turning lanes are intended to improve the safety and efficiency of traffic operations at these locations. Although the functions of turning lanes are well understood by highway engineers, there are no generally accepted warrants that define the circumstances under which the costs of constructing and maintaining turning lanes are justified. Therefore the University of Nebraska–Lincoln in cooperation with the Nebraska Department of Roads conducted research to develop warrants for turning lanes on uncontrolled approaches of rural intersections.

The specific objectives of the research were to *(a)* evaluate the safety and operational effects of turning lanes on uncontrolled approaches of intersections on rural two-lane highways, *(b)* develop a methodology for evaluating the cost-effectiveness of these lanes, and *(c)* use this methodology to develop guidelines for the cost-effective use of these lanes. The formulation of the cost-effectiveness methodology was based on a benefit-cost analysis approach. The benefits considered were the accident and operational cost savings provided to the road user by the turning lanes. The costs were those of constructing and maintaining the turning lane.

An analysis of intersection accidents on rural two-lane highways in Nebraska was conducted to determine the safety effects of turning lanes. In addition, a study of traffic operations on selected intersection approaches was conducted to further assess these safety effects. The procedure, findings, and conclusions of the traffic operations study are presented in this paper. The accident analysis, the formulation of the cost-effectiveness methodology, and the guidelines derived from its application have been reported elsewhere *(1)*.

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TYPES OF TURNING LANES

Three types of turning lanes are commonly used on two-lane highways in Nebraska: left-turn, right-turn, and fly-by lanes. A left-turn lane is usually constructed by widening the highway on both sides of the intersection to provide a protected lane for left-turning vehicles on one or both of the intersection approaches (Figure 1). A right-turn lane is normally con-

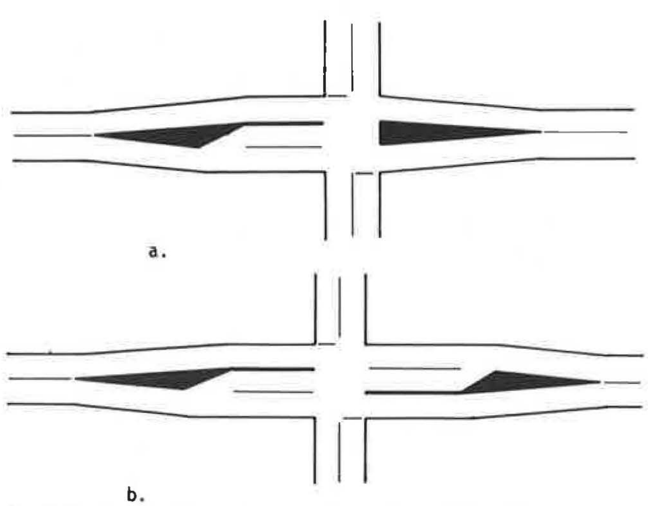

FIGURE 1 Left-turn lane configurations at four-leg intersection: *(a)* left-turn lane on only one approach; *(b)* left turn lanes on both approaches.

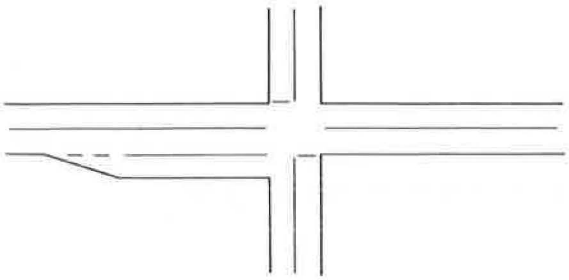

FIGURE 2 Right-turn lane configuration.

structed by widening the highway on the approach side of the intersection to provide a protected lane for right-turning vehicles on the intersection approach (Figure 2). A fly-by lane is usually provided at a T-intersection as a low-cost alternative to a left-turn lane. It is constructed by widening the highway on both sides of the intersection to provide a third lane through the intersection (Figure 3). This third lane is to be used by through vehicles to pass to the right of any left-turning vehicles that have slowed or stopped in the middle lane. When there are no left-turning vehicles, the through vehicles are to remain in the middle lane as they pass through the intersection. Thus, with a fly-by lane, the deceleration and storage of left-turning vehicles are not protected from the following through traffic as they are when there is a left-turn lane.

PROCEDURE

The traffic operations study was conducted on uncontrolled intersection approaches with and without turning lanes on rural two-lane highways in Nebraska. The study involved the time-lapse filming of traffic operations on the approaches. The films were analyzed to determine the effects of turning lanes on the safety of traffic operations.

Study Sites

The study sites were selected as representative of uncontrolled intersection approaches with and without turning lanes on rural two-lane highways in Nebraska. In the accident analysis phase of the research *(1)*, an inventory of all intersections on rural two-lane highways on the state highway system in Nebraska was compiled from the state highway map. The uncontrolled approaches of these intersections were classified according to type of turning lane present (none, left-turn, right-turn, or fly-by) and shoulder condition (paved or unpaved). Six approach categories were found:

Category	*Turning Lane*	*Shoulder*
I	None	Unpaved
II	None	Paved
III	Left-turn	Unpaved
IV	Left-turn	Paved
V	Right-turn	Unpaved
VI	Fly-by	Unpaved

The approaches without turning lanes and the approaches with left-turn lanes were found to have either paved or unpaved shoulders. However, the approaches with right-turn lanes and those with fly-by lanes were all found to have unpaved shoulders. In the case of an approach with a right-turn or fly-by lane on a highway that had paved shoulders, the turning lane was constructed on the paved shoulder and another paved shoulder was not added to the right of the turning lane.

From the inventory that was compiled in the accident analysis phase of this research, 36 approaches (6 from each of the 6 turning-lane categories used in the accident analysis) were selected as candidate sites for the conduct of the traffic operations study. However, field inspections of these sites revealed that only 14 of them had sufficient turning volumes and suitable vantage points from which to film. These 14 sites included at least one approach from each of the six turning-lane categories. Therefore, these 14 approaches were selected to be the study sites for the traffic operations study.

Eight of the study sites were approaches without turning lanes, five of which had no paved shoulders and three of which had paved shoulders. Three of the study sites had left-turn lanes, only one of which had a paved shoulder. Two of the study sites had right-turn lanes, and only one had a fly-by lane. The dimensions of the turning lanes at the study sites were in compliance with AASHTO *(2)* and Nebraska *(3)* geometric design standards. The route markings on all the study approaches were in accordance with standards for rural intersections of marked routes given in the *Manual on Uniform Traffic Control Devices for Streets and Highways* (MUTCD) *(4)*.

Data Collection

The traffic operations data were collected at the study sites by means of time-lapse photography. A 16-mm Automax Model 16-010 Cine-Pulse camera was used. The camera was operated at the film speed of 2 frames per second (fps). Depending on the particular location of the elevated vantage point from which the filming was conducted, a 25-, 50-, or 75-mm lens was used to provide a satisfactory field of view that extended from the far side of the intersection back upstream along the approach to a point 1,000 ft in advance of the intersection.

A total of 50 hr of traffic operations were filmed at the 14 study sites. Depending on the traffic volume, the length of the filming session at each site ranged from 2 hr to over 5 hr in order to obtain at least 100 turning movements of the appropriate type in each of the six turning-lane categories. The film used was 200-ft reels of 16-mm Kodak Ektachrome 7256 MS.

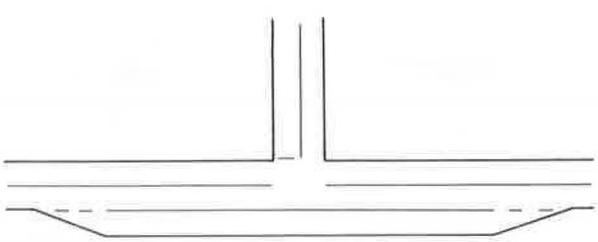

FIGURE 3 Fly-by lane configuration.

For filming at 2 fps, each reel of film provided about 1 hr 6 min of filming. Therefore, to minimize the amount of film used, the camera was only turned on when vehicles were on the approach.

Before a new reel of film was started, traffic cones were placed at 100-ft intervals on either side of the approach lane or lanes for a distance of 1,000 ft in advance of the intersection. From the vantage point, the focus of the camera was fixed to provide the necessary field of view. The approach was then filmed for a few seconds in the cine mode to provide a frame of reference for the subsequent analysis of the film. The traffic cones were then removed from the approach and time-lapse filming began.

Data Analysis

The time-lapse film was analyzed with a Lafayette 16-mm analyzer Model 300 projector, which was equipped with a frame counter and a range of viewing speeds both in forward and reverse. Each reel of film was projected onto a white paper screen. Once proper focus had been obtained, the film was stopped at the beginning of the reel, which showed the traffic cones placed on both sides of the approach lane in advance of the intersection. The locations of the traffic cones were marked on the screen and connected with straight lines to form a reference grid for the analysis of the film. Thus, a new reference grid was established for each reel of film analyzed.

Each reel of film was viewed several times at various speeds, both forward and in reverse, to record the position of each vehicle within the reference grid in each film frame (every ½ sec) as it traversed the approach. As a result, a time-space trajectory in three dimensions (time, distance, and lane placement) of each approach vehicle was obtained. In addition, the type and turning movement of each approach vehicle and the presence of an opposing vehicle at the intersection during its approach were recorded. Moreover, each film was viewed an additional time to record any traffic conflicts that occurred on the approach. These data were coded and input to the computer for further analysis.

Programs were written to compute the following measures of traffic operations safety on the approaches:

- Standard deviation of mean approach speed,
- Traffic conflict rate,
- Frequency of abnormal turning maneuvers, and
- Lane utilization factor.

Comparisons of these measures among the study approaches were used as indications of the relative safety of traffic operations on the approaches and measures of the safety effectiveness of the turning lanes.

Preliminary analysis of the speed data indicated that the mean speed of an approach vehicle over the 1,000 ft in advance of the intersection was affected by the turning movement and type of vehicle, the presence and turning movement of the vehicle ahead of the approach vehicle, and the presence of a vehicle on the opposing approach. Therefore, to eliminate the confounding effects of these factors on the comparisons of speed variance between approaches with and without turning lanes, stratified random samples were drawn from the original speed data so that the speed data used to make these comparisons were representative of similar traffic conditions. The standard deviations of mean approach speed were computed from these stratified random samples.

FINDINGS

Data were analyzed for more than 4,600 approach vehicles. The overall distributions of these data by turning movement and by vehicle type are shown in Table 1. The findings relative to each measure of effectiveness computed in the data analysis are presented in the following paragraphs.

Standard Deviation of Mean Approach Speed

In previous research *(5)* it has been found that, in general, the lower the speed differences between vehicles traveling on a

TABLE 1 OVERALL DISTRIBUTION OF TRAFFIC OPERATIONS DATA

Turning-Lane Category[a]	Turning Movement (%)			Trucks (%)	Sample Size
	Left-Turn	Through	Right-Turn		
I: None w/o shldr	13	82	5	30	1,222
II: None w/shldr	8	90	2	18	1,526
III: LT w/o shldr	26	73	1	35	370
IV: LT w/shldr	34	66	0	20	410
V: RT w/o shldr	0	53	47	16	560
VI: FB w/o shldr	13	87	0	17	780

[a]None - no turning lane; LT - left-turn lane; RT - right-turn lane; FB - fly-by lane;

w/o shldr - approaches without paved shoulders;

w/shldr - approaches with paved shoulders.

rural highway section, the lower the accident rates on the section. Therefore, the standard deviation of mean approach speed was used in this research to assess the safety of traffic operations on the intersection approaches. It was assumed for the purpose of this analysis that smaller standard deviations of mean approach speed were indicative of safer traffic operations on the intersection approaches.

Stratified Random Samples

As mentioned previously, in order to provide for meaningful comparisons between approaches with turning lanes and those without turning lanes, it was necessary to eliminate the confounding effects of those traffic conditions that affected mean approach speed by drawing from the original data random samples that were stratified according to the frequency with which these factors were present in the original data. Because, as shown in Table 1, the original sample sizes for the two approach categories without turning lanes were much larger than those of the four approach categories with turning lanes, the stratified random samples were drawn from these two larger samples in accordance with the distributions of data collected on the approaches with turning lanes and the corresponding percentage of trucks shown in Table 1.

Thus, in order to compare the standard deviation of mean approach speed on approaches with left-turn lanes and without paved shoulders with that on approaches without turning lanes and without paved shoulders, a stratified random sample was drawn from the original data for approaches without turning lanes and without paved shoulders so that the data used to compute the standard deviation of mean approach speed on left-turn lanes without paved shoulders were representative of the distribution of the data collected on the approaches with left-turn lanes and unpaved shoulders. Two additional stratified random samples were drawn from the original data for approaches without turning lanes and without paved shoulders, one to compare the standard deviation of mean approach speed of approaches with right-turn lanes and without paved shoulders and those without turning lanes and without paved shoulders and the other to compare approaches with fly-by lanes and without paved shoulders and approaches without turning lanes and without paved shoulders.

Only one stratified random sample was drawn from the original data for approaches without turning lanes and with paved shoulders. This sample was drawn in accordance with the distribution of the data collected on the approach with a left-turn lane and paved shoulder to compare approaches with left-turn lanes and paved shoulders and approaches without turning lanes and with paved shoulders.

Comparisons

The comparisons of the standard deviations of mean approach speed between approaches with turning lanes and those without turning lanes are shown in Table 2. These comparisons indicate that among the approaches without paved shoulders, the standard deviations of mean approach speed of through vehicles on approaches with turning lanes were statistically significantly lower than those of through vehicles on approaches without turning lanes, whereas those of left-turn and right-turn movements were not significantly different, according to the results of F-tests conducted at the 5 percent level of significance. The comparisons among approaches with paved shoulders, on the other hand, indicate that the standard deviation of mean approach speed of through vehicles on approaches with left-turn lanes was not significantly different from that of through vehicles on approaches without turning lanes. These findings suggest that the provision of turning lanes on approaches with-

TABLE 2 COMPARISONS OF STANDARD DEVIATIONS OF MEAN APPROACH SPEED

Comparison[b]	Turning Movement					
	Left-Turn		Through		Right-Turn	
	s	n	s	n	s	n
None w/o shldr	5.1	97	6.8	269	6.8	4
vs. LT w/o shldr	5.1	97	5.7[c]	269	5.0	4
None w/shldr	5.8	139	5.9	269	-	0
vs. LT w/shldr	5.6	139	6.0	269	-	0
None w/o shldr	-[d]	0	6.6	297	5.7	113
vs. RT w/o shldr	-	0	5.0[c]	297	5.6	113
None w/o shldr	5.5	102	7.0	678	-	0
vs. FB w/o shldr	5.7	102	5.2[c]	678	-	0

[a] s - standard deviation (mph); n - sample size.

[b] None - no turning lane; LT - left-turn lane; RT - right-turn lane; FB - fly-by lane; w/o shldr - approaches without paved shoulders; w/shldr - approaches with paved shoulders.

[c] Significantly lower than the standard deviation on similar approaches without turning lanes (α = 0.05)

[d] (-) - no data for this case.

TABLE 3 LEFT-TURN-LANE ACCIDENT REDUCTION FACTORS

Accident Type	Approaches Without Paved Shoulders	Approaches With Paved Shoulders
rearend + sideswipe	60%	10%
left-turn	-770%[a]	_[b]
right-turn	50%	0%

[a] "-" indicates an increase instead of a reduction.

[b] Infinite percentage increase in mean accident rate, because similar approaches without turning lanes had a zero mean accident rate for this type of accident.

out paved shoulders would improve the safety of traffic operations for the through vehicles on these approaches and not adversely affect the safety for the turning vehicles. However, the provision of left-turn lanes on approaches with paved shoulders would not improve the safety of these traffic operations. This observation was consistent with the findings of the accident analysis phase of the research *(1)*, shown in Table 3, in that the accident reduction factors for left-turn lanes on approaches with paved shoulders were lower than those for left-turn lanes on approaches without paved shoulders.

Traffic Conflict Rate

Any evasive actions by drivers to avoid collisions with other vehicles, other than normal braking and lane changing, were recorded as traffic conflicts. It was assumed that the lower the rate at which traffic conflicts occurred on an approach, the safer the traffic operations were on the approach. However, the rates of occurrence of traffic conflicts observed on the study approaches were so low that meaningful comparisons among the approaches were not possible.

The most frequently occurring traffic conflicts were on the approaches with left-turn or fly-by lanes. The most common one observed on approaches with left-turn lanes was between a left-turning vehicle and a through vehicle on the opposing approach. This conflict occurred when the sight distance between these two vehicles was obstructed by left-turning vehicles on the opposing approach. The most common conflict observed on the approach with a fly-by lane was between a through vehicle in a through lane and a through vehicle in the fly-by lane. This usually occurred when a second through vehicle arrived behind a left-turning vehicle and entered the fly-by lane before the through vehicle in front of it did. This conflict also occurred when a through vehicle attempted to use the fly-by lane to pass to the right of slower-moving through traffic.

Abnormal Turning Maneuvers

Another measure of effectiveness used to evaluate the safety of traffic operations on the study approaches was the percentage of turning vehicles that did not negotiate their turn in the normally expected manner along a curvilinear path without encroaching on shoulders, adjacent lanes, or both. Four types of turning maneuvers were defined as abnormal for the purpose of this analysis. The first was the wide turn, in which the turning vehicle did not complete its turn without swinging out and encroaching on shoulders, adjacent lanes, or both at the beginning or the end, or both, of its turn. The second was the straddle turn, in which the turning vehicle straddled the centerline of the highway from which it was turning for some distance before beginning its turn. This was usually done by left-turning vehicles to allow following through vehicles to pass; however, it was also observed to have been done by left-turning vehicles that were not being followed. By definition, the straddle turn did not apply to approaches with left-turn lanes.

The remaining two types of abnormal turns were only applicable to approaches with turning lanes: in the first, the angle turn, the turning vehicle cut diagonally across the turning lane without actually traveling in it, and in the second the turning vehicle never completely entered the turning lane to negotiate its turn.

Of course, the frequency of occurrence of abnormal turning maneuvers on the study approaches was not only influenced by the presence of turning lanes and paved shoulders but also by the turning radii provided on the approaches and the presence of other vehicles. However, for the purpose of this analysis, it was assumed that the lower the percentage of turning vehicles making abnormal turning maneuvers, the safer were the traffic operations on the approach.

Passenger Cars

The percentages of abnormal turning maneuvers by passenger cars for each approach category are shown in Table 4. On the basis of results of chi-square tests conducted at the 5 percent level of significance, the only statistically significant difference found in the abnormal-turning-maneuver percentages was between approaches with left-turn lanes and paved shoulders and those with left-turn lanes and unpaved shoulders. For some reason, on the approach with a left-turn lane and paved shoulders a significantly higher percentage of left-turning vehicles did not completely enter the left-turn lane to make their turns. Other than the possibility that the additional space provided on

TABLE 4 ABNORMAL TURNING MANEUVERS BY PASSENGER CARS[a]

Turning-Lane Category[b]	Turning Movement (%)									
	Left-Turn					Right-Turn				
	Abnormal				Normal	Abnormal				Normal
	Wide	Strad.	Angle	Inc.		Wide	Strad.	Angle	Inc.	
I: None w/o shldr	4.1	2.1	NA	NA	93.8	6.3	0	NA	NA	93.7
II: None w/shldr	0	2.1	NA	NA	96.1	3.2	0	NA	NA	96.8
III: LT w/o shldr	0	NA[d]	3.9	1.3[e]	94.8	33.3[f]	0	NA	NA	66.7
IV: LT w/shldr	3.0	NA	5.0	22.5[e]	69.5	-	-	NA	NA	-
V: RT w/o shldr	-[c]	-	-	-	-	4.1	0	1.7	7.5	86.7
VI: FB w/o shldr	1.1	0	NA	NA	98.9	-	-	NA	NA	-

[a] Wide - wide turn; Strad. - straddled center line; Angle - crossed turning lane diagonally; Inc. - never completely entered turning lane.

[b] None - no turning lane; LT - left-turn lane; RT - right-turn lane; FB - fly-by lane; w/o shldr - approaches without paved shoulder; w/shldr - approaches with paved shoulders.

[c] (-) - no turning movements of this type were made from approaches of this turning-lane category.

[d] NA - This type of abnormal turning maneuver is not applicable to this turning-lane category.

[e] Statistically significant difference between "LT w/o shldr" and "LT w/shldr" categories ($\alpha = 0.05$).

[f] Significantly higher than other turning lane categories ($\alpha = 0.05$).

the approach by the paved shoulder may have caused fewer drivers of these vehicles to perceive the need to completely enter the left-turn lane before making their turn, it was also possible that the path or turning radius provided by the left-turn channelization was not sufficient to encourage drivers to do so. However, the dimensions of the left-turn channelization were more than adequate for passenger cars according to AASHTO *(2)* and Nebraska *(3)* geometric design standards.

In the case of right-turning passenger cars, the only statistically significant difference found was in the percentage of right-turning vehicles making wide turns, which was significantly higher on approaches with left-turn lanes and unpaved shoulders than it was on those with right-turn lanes or without turning lanes. Nearly all of the wide turns made on the former approaches involved a right-turning vehicle that swung out to the left and encroached on the adjacent left-turn lane to begin its turn but did not encroach on the opposing lane of the intersecting highway to complete its turn. This higher percentage of wide turns did not appear to be due to shorter turning radii on these approaches, but instead to a greater inclination of right-turning drivers to encroach on adjacent left-turn lanes than on adjacent opposing lanes.

Trucks

The percentages of abnormal turning maneuvers by trucks for each approach category are shown in Table 5. Approaches with left-turn lanes had the lowest percentages of normal left-turn maneuvers because of the high percentages of left-turning trucks that did not completely enter the left-turn lanes to negotiate their turns. Instead, these trucks remained partly in the through lanes, apparently in order to maximize the radii of their turns. As was expected because of the longer turning radii of these turns, the results of chi-square tests conducted at the 5 percent level of significance indicated that the percentage of these abnormal turning maneuvers by trucks was significantly higher than that by passenger cars. In addition, it was noted that the percentage of wide-turns by left-turning trucks on approaches without turning lanes was not significantly different from that on approaches with left-turn lanes. However, because the dimensions of the left-turn channelization on these approaches were more than adequate for the trucks involved according to AASHTO *(2)* and Nebraska *(3)* geometric design standards, this finding suggested that the truck drivers merely took advantage of the additional space provided by the left-turn lanes to make a wide turn.

The abnormal-turning-maneuver percentages for right-turning trucks indicated that the provision of right-turn lanes on approaches without paved shoulders would significantly reduce the percentage of right-turning trucks that make wide turns. However, there was no significant difference in the percentage of normal right-turn maneuvers between approaches with right-turn lanes and those without right-turn lanes because of the percentage of right-turning trucks on approaches with right-turn lanes that made angle turns or failed to completely enter the right-turn lane in order to maximize the radii of their turns. In general it was also found, as expected, that because of the longer turning radii of trucks, the percentage of abnormal turning maneuvers by right-turning trucks was significantly higher than the corresponding percentage of abnormal turning maneuvers by right-turning passenger cars shown in Table 4.

Proper Lane Utilization

The final measure of effectiveness used to evaluate the safety of traffic operations on the study approaches was the percent-

TABLE 5 ABNORMAL TURNING MANEUVERS BY TRUCKS[a]

Turning-Lane Category[b]	Turning Movement (%)									
	Left-Turn					Right-Turn				
	Abnormal				Normal	Abnormal				Normal
	Wide	Strad.	Angle	Inc.		Wide	Strad.	Angle	Inc.	
I: None w/o shldr	3.4	0	NA	NA	96.6	28.5[e,f]	0	NA	NA	71.5
II: None w/shldr	6.7	0	NA	NA	93.3	-	-	NA	NA	-
III: LT w/o shldr	10.0	NA[d]	0	61.9[f]	28.1	-	-	NA	NA	-
IV: LT w/shldr	6.3	NA	15.4[f]	56.6[f]	21.7	-	-	-	-	-
V: RT w/o shldr	-[c]	-	-	-	-	6.9[e]	0	6.9[f]	14.8[f]	71.4
VI: FB w/o shldr	0	0	NA	NA	100.0	-	-	-	-	-

[a] Wide - wide turn; Strad. - straddled center line; Angle - crossed turning lane diagonally; Inc. - never completely entered turning lane.

[b] None - no turning lane; LT - left-turn lane; RT - right-turn lane; FB - fly-by lane; w/o shldr - approaches without paved shoulders; w/shldr - approaches with paved shoulders.

[c] (-) - no turning movements of this type were made from approaches of this turning lane category.

[d] NA - This type of abnormal turning maneuver is not applicable to this turning-lane category.

[e] Statistically significant difference between "None w/o shldr" and "RT w/o shldr" categories (α = 0.05).

[f] Significantly higher than the corresponding percentage for passenger cars shown in Table 5 (α = 0.05).

age of turns that were made from the proper approach lane. On the approaches without turning lanes, all turns should have been made from the through lane. On the approaches with left-turn lanes, the left turns should have been made from the left-turn lane and the through and right turns should have been made from the through lane. On approaches with right-turn lanes, the left turns and through movements should have been made from the through lane and the right turns should have been made from the right-turn lane. On the approach with a fly-by lane, the left turns should have been made from the through lane and the through vehicles should have also used the through lane unless they arrived behind a left-turning vehicle, in which case they should have used the fly-by lane. Because the fly-by lane observed was at a T-intersection, there were no right turns made from the study approach. Turns made from shoulders, opposing traffic lanes, or the wrong approach lane were considered to be potential safety hazards. Therefore, it was assumed for the purpose of this analysis that higher percentages of proper lane utilization were indicative of safer traffic operations on the study approaches. The lane utilization by turning movements for each approach category is shown in Table 6.

Left Turns

In the case of left turns, only a few were not made from the through lanes on approaches without left-turn lanes. All the left turns not made from these through lanes were made from the opposing traffic lane in order to allow a following through vehicle to pass by on the right without delay. Such improper turns were not observed on the approaches with left-turn lanes. However, the results of a chi-square test conducted at the 5 percent level of significance indicated that a significantly lower percentage of left-turning vehicles used the left-turn lane on the approach with the paved shoulders than on the approach without paved shoulders. Instead, these vehicles turned left from the through lanes, in some cases even when being followed by a through vehicle. Therefore, perhaps the additional space provided by the paved shoulder caused the drivers of these left-turning vehicles not to perceive the need to use the left-turn lane. Although some of these drivers may not have been sure which way they needed to turn in order to reach their destination and therefore did not pull off into the turn lane, this was not apparent from the film analysis, because all of these turns were made without hesitation.

Through Movements

All of the through movements on approaches with left-turn or right-turn lanes were made from the through lanes. On the approaches without turning lanes, a few of the through movements were not made from the through lane. On the approaches with no turning lanes and unpaved shoulders, these improper through movements were made in the opposing traffic lane in order to pass on the left of a right-turning vehicle. On the approaches with no turning lanes and paved shoulders, these improper through movements were made on the shoulder in order to pass to the right of a left-turning vehicle.

On the study approach with a fly-by lane, about 5 percent of the through movements made in the fly-by lane were not necessary because there was no left-turning vehicle ahead of the through vehicle. Another 5 percent of these through movements were improper in that the through vehicle used the fly-by lane to pass to the right of a slower-moving through vehicle ahead. Only about 1 percent of the through movements made in the through lane should have been made in the fly-by lane in order to avoid slowing for a left-turning vehicle ahead and thus,

TABLE 6 LANES USED BY TURNING MOVEMENTS[a]

Turning-Lane Category[b]	Turning Movement (%)											
	Left-Turn				Through				Right-Turn			
	TL	Thru	Shldr	Opp	TL	Thru	Shldr	Opp	TL	Thru	Shldr	Opp
I: None w/o shldr	NA[d]	98.1	NA	1.9	NA	99.9	NA	0.1	NA	100.0[f]	NA	0
II: None w/shldr	NA	96.6	0	3.4	NA	99.7	0.3	0	NA	77.4[f]	22.6	0
III: LT w/o shldr	100.0[e]	0[e]	NA	0	0	100.0	NA	0	0	100.0	NA	0
IV: LT w/shldr	95.7[e]	4.3[e]	0	0	0	100.0	0	0	-	-	-	-
V: RT w/o shldr	-[c]	-	NA	-	0	100.0	NA	0	99.0	1.0	NA	0
VI: FB w/o shldr	0	100.0	NA	0	25.0[g]	75.0[h]	NA	0	-	-	NA	-

[a] TL - turning lane; Thru - through lane; Shldr - shoulder; Opp - opposing traffic lane.

[b] None - no turning lane; LT - left-turn lane; RT - right-turn lane; FB - fly-by lane; w/o shldr - approaches without paved shoulders; w/shldr - approaches with paved shoulders.

[c] (-) - no turning movements of this type were made from approaches of this turning-lane category.

[d] NA - Use of this lane is not possible on approaches in this turning-lane category.

[e] Statistically significant difference between "LT w/o shldr" and "LT w/shldr" categories ($\alpha = 0.05$).

[f] Statistically significant difference between "None w/o shldr" and "None w/shldr" categories ($\alpha = 0.05$).

[g] About 5% of these through vehicles unnecessarily usedthe fly-by lane, and another 5% of them used the fly-by lane to pass another through vehicle.

[h] Approximately 1% of these through vehicles should have used the fly-by lane.

in these instances, negating the safety and operational benefits to be derived from use of the fly-by lane.

Right Turns

All right turns were made from the through lanes on the approaches with no turning lanes and unpaved shoulders. However, on the approaches with no turning lanes and paved shoulders, more than 20 percent of the right turns were made from the shoulders, usually to allow following through vehicles to pass by on the left. On the approaches with left-turn lanes, all right turns were made from the through lanes. But on the approaches with right-turn lanes, only 99 percent of the right turns were made from the right-turn lanes. The remaining 1 percent was made by trucks from the through lanes in order to maximize the radius of their turns, even though, according to AASHTO *(2)* and Nebraska *(3)* geometric design standards, the turning radii provided were adequate for the trucks involved. In some cases, through vehicles were following these trucks and the safety and operational benefits of the right-turn lanes were thus negated.

CONCLUSION

In general, the results of the traffic operations study were found to support the findings of the accident analysis phase of the research, which has been reported elsewhere *(1)*. The measures of effectiveness computed indicated that the provision of turning lanes on the uncontrolled approaches of intersections on rural two-lane highways improved the safety of traffic operations on these approaches, especially those without paved shoulders. Comparisons of the standard deviations of the speed on the study approaches indicated that the provision of a turning lane (left-turn, right-turn, or fly-by lane) on an approach that does not have paved shoulders would improve the safety of traffic operations on that approach but that the provision of a left-turn lane on an approach that already has a paved shoulder would not.

The turning-maneuver and lane-utilization analyses that were conducted indicated that this failure of left-turn lanes to improve the safety of traffic operations on approaches that already had paved shoulders was due, at least in part, to the following driver behavior:

1. On approaches with left-turn lanes and paved shoulders, a high percentage of drivers (27.5 percent of passenger-car drivers and 78.3 percent of truck drivers) did not completely enter or properly utilize the turning lane to make a left turn;
2. On approaches with left-turn lanes and paved shoulders, a statistically significant percentage of drivers (4.3 percent) did not use the left-turn lane to make a left turn but instead turned left from the through lane; and
3. On approaches without a left-turn lane but with a paved shoulder, the shoulder was found to function as a fly-by lane for some through vehicles that were following left-turning vehicles and as a right-turn lane for right-turning vehicles.

These observations of driver behavior suggest that special attention should be given to the design, signing, and marking of left-turn lanes on approaches with paved shoulders in order to eliminate their improper use by drivers and encroachments by left-turning vehicles into adjacent through lanes, both of which negate the safety and operational benefits to be realized by the provision of left-turn lanes. Also, highway engineers should recognize that paved shoulders on approaches to intersections of marked rural highway routes are used by drivers as turning lanes so that these shoulders will be properly designed, both structurally and operationally, for this purpose.

The traffic operations study also revealed two potential operational problems with left-turn lanes and fly-by lanes that have been observed by others *(6, 7)*. The most common traffic conflict found on approaches with left-turn lanes was between a left-turning vehicle and a through vehicle on the opposing approach. This conflict occurred when the sight distance between these two vehicles was obstructed by left-turning vehicles on the opposing approach. Therefore, in providing left-turn lanes, the highway designer should avoid creating this problem by offsetting opposing left-turn lanes if necessary to provide adequate sight distance for left-turning and opposing through vehicles *(6)*.

The analyses of the traffic conflict and lane utilization data indicated the potential for improper use of the fly-by lane by through vehicles attempting to pass other through vehicles. To reduce this potential, the highway designer should give special attention to the proper design and application of fly-by lanes as recommended by Buehler *(7)*, using short approach lengths and long departure lengths of the fly-by lanes and not using fly-by lanes as substitutes for left-turn lanes at locations with higher traffic volumes.

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Comparison of Different Procedures for Evaluating Speed Consistency

RUEDIGER LAMM, JOHN C. HAYWARD, AND JEFFREY G. CARGIN

The need for achieving operating-speed consistency on two-lane rural highways through consistent horizontal alignment is discussed. One American and two European methods for evaluating horizontal alignment consistency are compared: a graphical speed-profile technique proposed for use in the United States, a theoretical speed model utilized by the Swiss highway design community, and a German procedure using a design parameter known as the curvature change rate. The results of the comparison of the three approaches show that although at times substantially different speed values are obtained from each method, the fundamental results necessary to evaluate alignment consistency are basically the same. On the basis of this comparison, it appears that the curvature-change-rate method is the most convenient for predicting changes in operating-speed profile along a rural roadway brought about by inconsistencies in horizontal alignment.

According to Cleveland et al. *(1)*,

> Two-lane rural highway safety is an issue of pressing national concern. It has been identified as the highest priority research need in the area of responsibility of the TRB Committee on Geometric Design. These roads constitute approximately 4 million km (2.5 million miles) or 63 percent of the highways in the United States and are the locations of about 50 percent of all highway fatalities. They have the highest accident rate of any class of rural highway, with fatal and injury vehicle-mile exposure accident rates (VMER) consistently being four to seven times higher than those on rural interstate highways.

More than 60 percent of the total accidents and about 80 percent of the fatalities on two-lane rural highways may be indirectly attributed to improper speed estimation. Although human factors may be identified as a major cause in all accidents, the driver's frame of mind and physical condition are virtually impossible to control from a design standpoint. Besides alcohol abuse, absence of seat-belt use, and poor judgment at intersections, most of the errors due to excessive speed occur with reference to road design. Young drivers aged 15 to 24 are especially endangered, in large part because of their lack of driving experience. This age group represented about 36 percent of all fatalities in the United States in 1982 *(2–6)*.

Many of these speed errors may be related to inconsistencies in horizontal alignment that cause the driver to be surprised by sudden changes in the road's alignment, to exceed the critical speed of a curve, and to lose control of the vehicle. These inconsistencies can and should be controlled by the engineer when a roadway section is designed or improved *(7, 8)*.

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Many experts believe that abrupt changes in operating speed lead to accidents on two-lane rural roads and that these speed inconsistencies may be largely attributed to abrupt changes in horizontal alignment *(9–12)*. Approximately $2 billion from federal and state sources is spent annually on the resurfacing, restoration, and rehabilitation of two-lane rural roads in the United States. This program is intended to extend the useful service life of these highways without the addition of many costly geometric redesigns. New designs and major reconstruction are not included in this significant expenditure *(13)*. Considering the magnitude of this annual investment, it is clear that a convenient method for locating alignment inconsistencies would provide a first step in the improved allocation of these resources by identifying the need for improved horizontal alignment. Providing longer road sections with relatively consistent alignment and thereby a consistent driving behavior is an important step in reducing critical driving maneuvers, thereby obtaining less hazardous road sections and enhancing traffic safety on two-lane highways.

Methods to improve highway alignment consistency have existed in several western European countries for more than the last decade. Similar procedures have been proposed for use in the United States but are not yet considered standard. One American and two European methods are discussed in the following pages: an operating-speed concept proposed by Leisch and Leisch *(9)*, a theoretical speed model used in the Swiss design standard *(14, 15)*, and a German design procedure related to a parameter known as the curvature change rate *(15–18)*.

BACKGROUND

Many studies have been conducted that focus on obtaining a more consistent design and the effect of inconsistencies on traffic safety. A survey of the literature by Hayward *(12)* suggests that easing a few sharp curves may have a much greater effect on safety improvement than easing more gentle curves.

Among other results, the following statement was made by the New York State Department of Transportation in 1983 *(19)*:

> Among improvement types showing large accident reductions and fair safety benefit cost ratios are horizontal alignment changes. Horizontal alignment improvements at 15 sites resulted in overall accident reductions of 45 percent and fatal/injury accident reductions of 42 percent. Efforts must be made to bring as many two lane rural roads to an acceptable level of operating speed consistency as possible.

The increased use of operating speed as a preferred criterion over design speed was also noted in the 1977 AASHTO Guidelines for resurfacing, restoration, and rehabilitation

(RRR) projects *(20)*: "The desirable design should accommodate the current running speed and a minimum design speed should not be established."

ALTERNATIVE DESIGN STRATEGIES

Efforts to define a systematic process for evaluating horizontal design and its subsequent impact on operating-speed profiles have been proposed for the United States; such a process is in use in Switzerland and Germany. Each is briefly described in the following sections.

Leisch Method *(9)*

Leisch and Leisch have suggested that using design speed alone as the control for design may lead to undesirable geometry. Even though design speed has been used for several decades to determine allowable horizontal alignment, it is possible to design certain inconsistencies into highway alignment. At low and intermediate design speeds, the portions of relatively flat alignments interspersed between the controlling curvilinear portions may produce operating-speed profiles that may exceed the design in the controlling sections by substantial amounts.

To overcome this weakness in current practice, Leisch and Leisch have suggested a new concept in the definition and application of design speed. The overall objective is to design for driver expectations and to comply with inherent driver characteristics to achieve operational consistency and improve driving comfort and safety.

The Leisch method involves using a speed-profile technique to achieve consistency in the horizontal and vertical alignments. They suggest the use of the "10-mph rule" as a design principle applied in specific situations as follows:

- Within a given design speed, potential average passenger-car speeds generally should not vary more than 10 mph (~16 km/hr).
- A reduction in design speed, where called for, normally should not be more than 10 mph.
- Potential average truck speeds generally should not be more than 10 mph below average passenger-car speeds at any time on common lanes.

This procedure consists of determining the average running speeds on horizontal curves in accordance with the low-volume relations of average running speed to design speed contained in the AASHTO guidelines *(21, 22)* and combining these with a series of nomographs to determine the amount of acceleration and deceleration for passenger cars and trucks. Both the horizontal and vertical alignments are taken into consideration, and a resulting speed profile is developed for the road section. Comparing the speed profile with the 10-mph rule, inconsistencies in the profile may be located and the design may be adjusted to eliminate them. The Leisch method is one of the first methods developed in the United States that may be used for evaluating consistency in the horizontal and vertical alignments of a roadway. A more detailed discussion and an example of this procedure will be given later in this discussion.

Swiss Method *(14, 15)*

The design speed concept as it is defined in Switzerland is not directly comparable with that used in the United States. Because the level of design and construction of a certain type of road is not fixed in Switzerland, ranges of design speed are assigned to each road type. Criteria that must be considered by the designer when a design speed is selected include the importance of the road, the traffic volume and mix, and the characteristics of the topography. The selected design speed is then used to determine minimum design values in a manner similar to that found in U.S. guidelines.

However, in addition to the design speed concept, the Swiss use a theoretical speed model to analyze the consistency of the horizontal alignment. This procedure is similar to the Leisch method in that it utilizes a speed-profile diagram to detect abrupt changes in what the Swiss determine to be the project speed. (Project speed is comparable with operating speed in the United States).

The project speed is modeled from geometric design of the horizontal and vertical alignments. It is expected to predict the maximum speed to be found on a certain roadway section. This project speed (not design speed) serves as a test speed to assess adequate sight distances and to evaluate adequate superelevation rates in cases when the project speed is higher than the design speed.

Standard values for the project speed have been determined through field research and are tabulated for different radii. The speed is considered constant over the length of the curve. Changes in the project speed between two successive curves, or between a curve and a tangent, are normally not allowed to exceed ~12 mph (20 km/hr), but for project speeds of less than 45 mph (70 km/hr) a speed change of less than ~6 mph (10 km/hr) is desirable.

The Swiss have developed a formula for calculating the "transition length," which is the distance required for acceleration or deceleration of a vehicle as it approaches or leaves a curve based on the speed difference between two curves or between a curve and a tangent. Unacceptable ranges for these transition lengths are also tabulated.

A speed diagram is used to graphically locate inconsistencies in the speed profile and thereby inconsistencies in the horizontal alignment. Because the Swiss make several simplifying assumptions, this method is easy to use and similar conceptually to the Leisch method. A more detailed discussion of this procedure and an example illustrating its use will be given in the next section.

German Method *(15–18)*

Highway design speed as applied in Germany depends on many issues, including environment and economic conditions, function of the road network, travel purposes, quality of traffic flow, road category, topography, and so on. As in the United States, the design speed is used to determine minimum design values. The Germans acknowledge that the design speed influences many roadway characteristics and therefore decisively affects traffic safety, quality of the traffic flow, and the local economy. German practice requires that constant design speed be applied to long lengths of road sections or particular roadway classifications.

In addition to the design speed, German designers use operating speeds to control design standards. The operating speed as defined in Germany corresponds to the 85th-percentile speed of passenger cars under free-flow conditions for clean, wet road surfaces. Because the 85th-percentile speed is normally higher than the design speed, the operating speed is a value used instead of design speed to determine adequate superelevation rates and necessary stopping sight distances. Use of this higher value builds in an additional factor of driving safety for roadway elements.

In contrast to the Leisch and Swiss methods, a different approach toward achieving consistency in horizontal alignment is taken in the German design guidelines. Instead of working with single curves and speed profiles, the Germans use a parameter called "curvature change rate" (CCR) to describe overall roadway homogeneity and to prevent abrupt (and unsafe) transitions in operating speeds between local homogeneous sections of roadways. CCR is defined as the absolute sum of the angular changes in the horizontal alignment divided by the length of the highway section.

It has been found through field observations in Germany that the operating speed remains relatively constant over the length of sections with similar characteristics and that this operating speed is strongly correlated to CCR *(7, 18, 23)*. Lengths and radii of all circular curves and lengths of all transition curves and tangents within the section may be used to compute CCR. A nomograph relating CCR to operating speed may be used to predict the operating speed of the section.

Currently, German design guidelines *(16)* require that the predicted operating speed within any given section not exceed the design speed of that section by more than ~12 mph (20 km/hr). Furthermore, a limit on the permissible section-to-section difference in the operating speeds not to exceed ~6 mph (10 km/hr) ensures operational consistency and provides a balanced design. If these conditions are not met for any particular section, the design of the horizontal alignment must be adjusted.

A more detailed discussion and an example illustrating the use of the procedure will be given in the next section.

EVALUATING OPERATING-SPEED CONSISTENCY

The differences among the speed-profile techniques of Leisch, the Swiss, and the Germans are interesting to compare. Example applications of the three methods on a single roadway section (Figure 1) demonstrate the difference in how each is used to identify operating-speed inconsistencies. The common example roadway shown in Figure 1 is a two-lane main primary rural road with lanes 10 ft wide, and for simplicity it is assumed that the vertical alignment of the entire section is level. The alignment before point *A* traveling west to east is assumed to be a long tangent section. Only the operating-speed profiles for passenger cars will be constructed.

Example of Leisch Method

The basic characteristics of the speed-profile technique proposed for use in the United States are as follows:

1. The profile is based on low-volume free-flow conditions, using the average running speeds of traffic under favorable roadway conditions (daylight, good weather, etc.).
2. The top average running speed of passenger cars for a given highway type may be found in Table 1. These speeds are based on open, near-level, and straight highways outside the influence of any other geometric constraints.
3. The average running speeds through horizontal curves are taken from Figure 2, which was developed according to low-volume relations of average running speed to design speed adapted from the 1965 AASHO geometric design policy *(21)*.
4. Deceleration and acceleration distances have to be taken from additional nomographs that were adapted and extrapolated from the 1965 AASHO guidelines for acceleration and deceleration at intersections and interchanges *(21)*.

By using Table 1 and Figure 2 of the Leisch method *(9)*, the following information may be determined:

- Top average speed for the highway: 60 mph (100 km/hr) (Table 1)

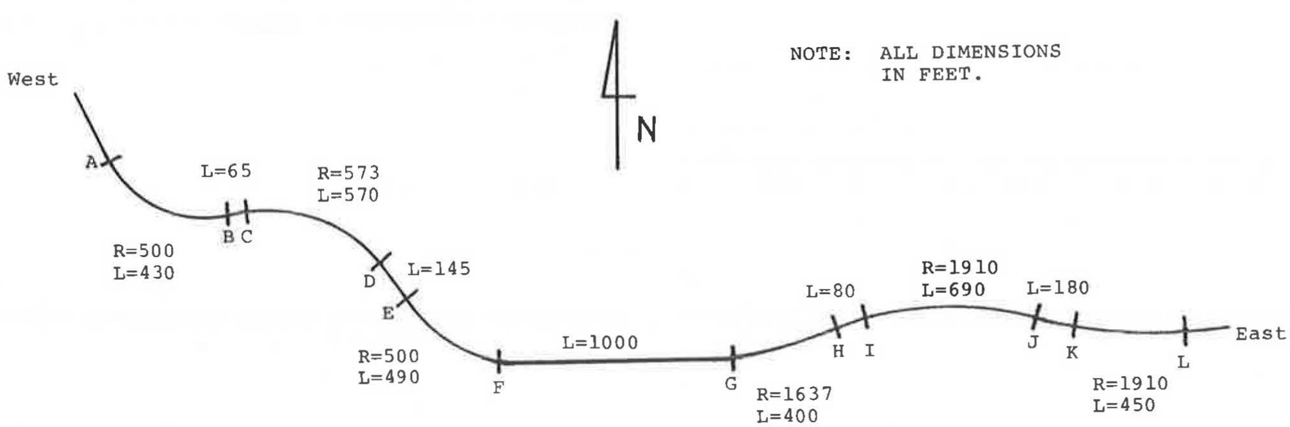

FIGURE 1 Horizontal alignment of roadway section to be examined for operating-speed consistency.

TABLE 1 TOP AVERAGE SPEED OF PASSENGER CARS ON VARIOUS TYPES OF HIGHWAYS FOR USE IN LEISCH METHOD *(9)*

Type of Facility	Highway Quality and Condition: Favorable		Moderate	
	mph	(km/h)	mph	(km/h)
Rural Highways				
Interstate	65	(100)	60	(95)
Primary - Main	60	(95)	55	(90)
Primary - Intermediate	55	(90)	50	(80)
Secondary	50	(80)	45	(70)
Urban Highways				
Interstate	60	(95)	55	(90)
Arterial - Main	50	(80)	45	(70)
Arterial - Intermediate	45	(70)	40	(65)
Secondary	40	(65)	35	(55)

Representative of low-volume, free-flowing conditions on open, near level and straight highways.

- Speed of Curve *AB*: 37 mph (~60 km/hr) (Figure 2)
- Speed of Curve *CD*: 39 mph (~60 km/hr) (Figure 2)
- Speed of Curve *EF*: 37 mph (~60 km/hr) (Figure 2)
- Speed of Curve *GH*: 58 mph (~90 km/hr) (Figure 2)
- Speed of Curve *IJ*: 60 mph (Figure 2)
- Speed of Curve *KL*: 60 mph (Figure 2)

Because this method uses a sophisticated technique for determining acceleration and deceleration distances, certain assumptions have to be made about speed reductions approaching a curve, sight distances, and topography (in this case, level terrain) when these nomographs are used *(22)*. These assumptions will not be discussed in detail in this paper, but it should be noted that different users of this procedure may arrive at slightly different values. However, the basic form of the speed profile will remain approximately the same; it is shown in Figure 3 for west–east travel and Figure 4 for east–west travel.

On examination of Figure 3 (west–east travel) it may be seen that there is an unacceptable break in the operating speed at point *A*. The speed difference is 23 mph (60 mph – 37 mph), which is greater than the recommended limit of 10 mph for this method. Also noticeable from the diagram is that the distance required to accelerate from 37 mph to 60 mph is longer than the remaining length of the section, points *F* to *L*. More information is necessary about the alignment after point *L* to determine whether the assumed maximum speed of 60 mph will actually be reached.

It should be noted that the speed profile in Figure 3 is valid

IMPERIAL UNITS

V-DESIGN SPEED, MPH	30				40				50				60			70			80
CORRESPONDING AVERAGE RUNNING SPEED, MPH																58	60	62	64
													52	54	56	58	60	62	64
									44	46	48	50	52	54	56	58 →			
					36	38	40	42	44	46	48	50	52	54 →					
	28	30	32	34	36	38	40	42	44	46	48 →								
D_C-MAX. DEGREE OF CURVE*	23.0	19.1	16.4	14.1	12.5	10.8	9.5	8.5	7.5	6.8	6.2	5.6	5.0	4.5	4.0	3.5	3.1	2.8	2.5
R_C-MIN. RADIUS, FEET	250	300	350	400	460	530	600	680	760	840	930	1030	1140	1280	1440	1630	1820	2030	2240

PERMISSIBLE AVERAGE RUNNING SPEEDS ON CURVES OF GIVEN RADII AT LOW-VOLUME, FREE-FLOW CONDITIONS - APPLICABLE TO PASSENGER CARS

*D_C = 5729.5780 ÷ R_C (Based on central angle subtending 100-foot arc)

Tabular values are based on an average maximum superelevation rate of .08

→ For a designated or estimated design speed, any larger radii beyond the arrow are assumed to have the same average running speed as at the arrow.

FIGURE 2 Speed-curvature relationships for use in Leisch method *(9)* [adapted from AASHO guidelines *(21)*].

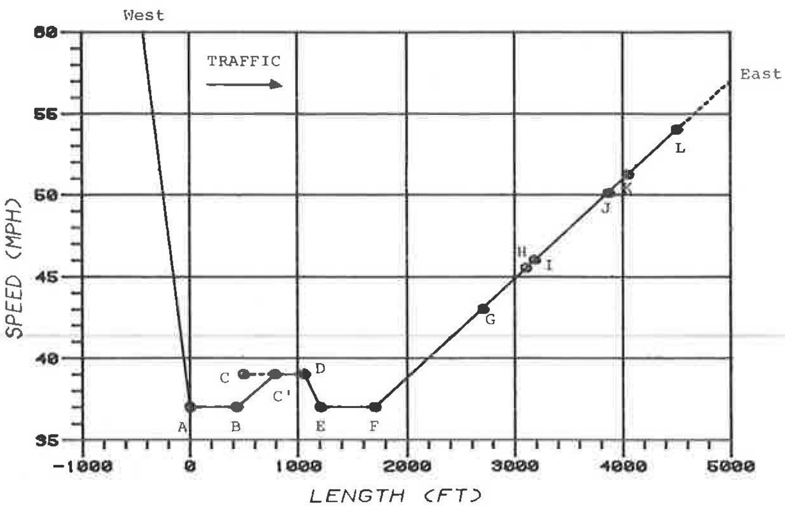

FIGURE 3 Speed profile resulting from application of Leisch method on roadway section of Figure 1 for west–east travel.

only for the direction of travel shown, because the acceleration and deceleration rates are different. Therefore, in order for the analysis to be complete, this same procedure must also be used to construct a similar speed diagram for the east–west direction of travel (Figure 4). In this direction, a speed difference of 22 mph (59 mph – 37 mph) occurs between points *G'* and *F*, which also exceeds the 10-mph limit.

By using this procedure, the speed changes before point *A* for west–east travel and point *F* for east–west travel are identified as critical areas in which the horizontal alignment causes an inconsistency in the speed profile. These critical locations should be investigated further to determine whether any corrective action should be taken.

Example of Swiss Method

Switzerland employs a speed model to examine consistency in horizontal alignment and to recognize dangerous breaks or transitions in the speed profile brought on by changes in horizontal alignment. The speed model represents the theoretical course of the project speed as a function of horizontal curvature. Several assumptions are made to simplify the procedure considerably, including the following:

1. The driver selects the project speed for a curve on the basis of the radius of the curve, and this speed is considered to remain constant throughout the curve. The project speed for any given radius may be taken from Table 2. For radii falling between the values in the table, the higher value should be chosen (not interpolated). Also, the horizontal lines in the table indicate the maximum allowable speed corresponding to the speed limit for each type of road. For example, rural roads have a maximum speed of 62 mph (100 km/hr).
2. The speed in tangents and transition curves corresponds to the posted speed limit (the horizontal lines in Table 2).
3. Decleration ends at the beginning of the circular curve.
4. Acceleration begins at the end of the circular curve.

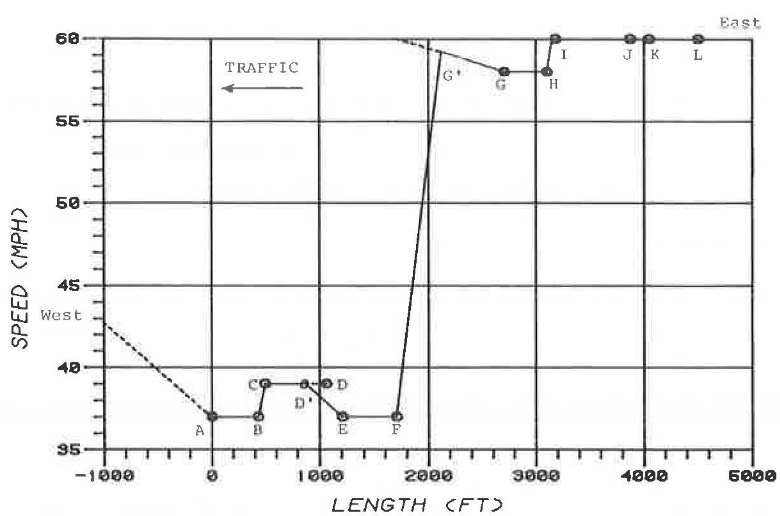

FIGURE 4 Speed profile resulting from application of Leisch method on roadway section of Figure 1 for east–west travel.

TABLE 2 RELATIONSHIP BETWEEN RADII AND PROJECT SPEED FOR USE IN SWISS METHOD *(14, 15)*

Radii [m]	Radii [ft]	Project Speed [km/h]	Project Speed [mph]	
45	148	40	25	
60	197	45	28	Urban
75	246	50	31	Roads
95	312	55	34	
120	394	60	37	
145	476	65	40	
175	574	70	43	
205	672	75	46	
240	787	80	50	Rural
280	918	85	53	Roads
320	1050	90	56	
370	1214	95	59	
420	1378	100	62	
470	1542	105	65	
525	1722	110	68	
580	1902	115	71	Interstate
650	2132	120	74	
710	2329	125	78	
>780	>2558	130*	81	

*120 km/h since January 1, 1985.

5. Acceleration and deceleration are considered to be equal and constant at a rate of a = ~2.6 ft/sec^2 (0.8 m/sec^2); thus one speed diagram is sufficient for both directions of travel.

6. The distance traveled during acceleration and deceleration, known as the transition length, may be taken from Figure 5 by using the project speeds between two consecutive design elements for the transition between two curves or between a curve and a tangent.

Because the relationships between project speed and radius are given in Table 2, a theoretical speed profile with corresponding transition lengths from one design element to the other may be established. Limits of maximum differences in the project speeds and ranges of unallowable, or avoidable, transition lengths between successive design elements with different project speeds (Figure 5) ensure operational consistency and provide a balanced horizontal design.

Using the procedures and assumptions noted earlier, the speed profile in Figure 6 may be constructed, which is related to the horizontal alignment in Figure 1. From Table 2 the project speeds of Curves *AB*, *CD*, and *EF* are each found to be ~43 mph (70 km/hr), and in Curves *GH*, *IJ*, and *KL* the speeds are found to be the maximum value of ~62 mph (100 km/hr), which is the speed limit for rural roads in Switzerland. This is comparable with the top average running speed used in the Leisch method.

The required acceleration and deceleration lengths may be taken from Figure 5 when the appropriate project speeds are known. For the acceleration after point *F*, Vp_1 = ~43 mph (70 km/hr) and Vp_2 = ~62 mph (100 km/hr), so the required distance is approximately 800 ft (245 m); thus, a speed ~62 mph would be reached at point *F'*. This would also be the distance required for the deceleration before point *A*, because the speeds involved are the same in both cases. The transition length of 800 ft resulting from the two project speeds falls into the range of transition lengths that should be avoided (Figure 5) and would indicate that a transition between these two speeds would cause an inconsistency to occur.

The short tangent sections *BC* and *DE* produce only a negligible amount of acceleration, so the speeds in these sections are assumed to be the same as those on the curves surrounding them.

In addition, examination of Figure 6 reveals that speed breaks of 19 mph (62 mph − 43 mph) occur before point *A* for

V_{p1} (km/h) resp. V_{p2} / V_{p2} (km/h) resp. V_{p1}	40	45	50	55	60	65	70	75	80	85	90	95	100	105	110	115	120	125	130
40																			
45	20																		
50	45	25																	
55	70	50	25																
60	95	75	55	30															
65	125	105	85	60	30														
70	160	140	115	90	65	35													
75	195	175	150	125	100	70	35												
80	230	210	190	165	135	105	70	35											
85	270	250	230	205	175	145	110	75	40										
90	315	295	270	245	220	185	155	120	80	40									
95	360	340	315	290	260	230	200	165	125	85	45								
100	405	385	360	335	310	280	245	210	175	135	90	45							
105	455	435	410	385	360	330	295	260	225	185	140	95	50						
110	605	485	465	440	410	380	345	310	275	235	195	145	100	50					
115	660	540	515	490	465	438	400	365	330	290	245	205	155	105	55				
120	615	600	575	550	520	490	460	425	385	345	305	260	210	165	110	55			
125	675	665	635	610	580	550	515	480	445	405	365	320	270	220	170	115	60		
130	740	715	695	670	640	610	580	545	505	465	425	380	335	285	230	175	120	60	

Transitions of project speeds that should be avoided are listed in the middle section of the nomograph

Not allowable ranges of decelerations are listed in the lower left section of the nomograph

FIGURE 5 Required transition lengths for acceleration and deceleration for use in Swiss method *(14, 15)*.

west–east travel and before point *F* for east–west travel. These values are greater than the suggested speed change limit of ~12 mph (20 km/hr) in the Swiss method. The critical sections identified through the Swiss procedure occur at the same locations as they do in the Leisch method.

However, several differences are immediately obvious between this technique and the Leisch method. First, the speed values in the first curved section (*AF*) are 4 to 6 mph higher in the Swiss method than those obtained from the Leisch method (see Table 3). Second, there is a substantial difference in the distances required for acceleration after point *F*. With the Leisch method the acceleration is not completed within the section being examined, whereas in the Swiss method acceleration ends before curve *GH* is reached. This is a major discrepancy between the two procedures, due in large part to the fact that both use theoretical acceleration rates, which should be tested under actual driving conditions. Therefore, determining accurate acceleration and deceleration rates should be the objective of a future study.

Example of German Method

German designers use different techniques to guide the design of horizontal alignment. These include policies that control successive curves, lengths of tangents, and consistency in horizontal alignment in addition to controls on CCR.

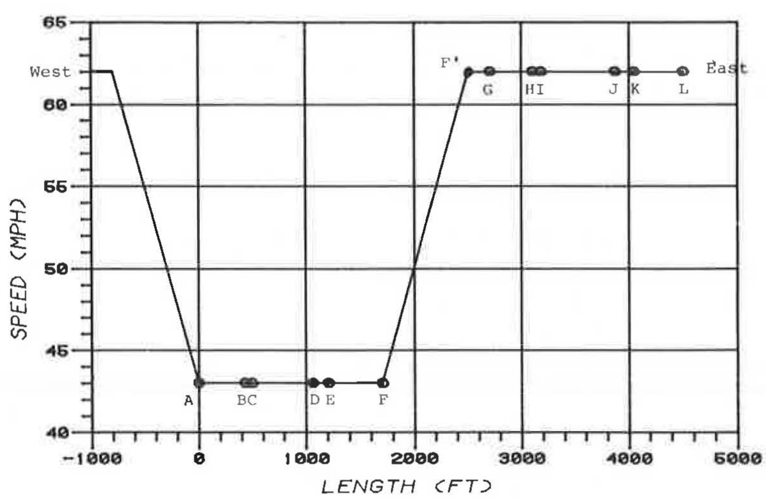

FIGURE 6 Speed profile resulting from application of Swiss method on roadway section of Figure 1 for both directions of travel.

TABLE 3 COMPARISON OF OPERATING-SPEED VALUES (MPH) OBTAINED FROM THREE METHODS

Method	AB	CD	EF	GH	IJ	KL	
	Curve						
Leisch W-E	37	39	37	~44	~47	~53	mph
Leisch E-W	37	39	37	58	60	60	mph
Swiss	43	43	43	62	62	62	mph
German	41	41	41	51	51	51	mph

Nomographs in the design manuals provide guidance on safe combinations of successive curves. The radii of successive curves must fall within acceptable ranges, which increase as the curves become flatter.

Tangent sections between curves are limited by the design speed. The length of tangent (in meters) between two curves cannot exceed 20 times the design speed (in kilometers per hour) of that roadway. In this manner, long tangents are controlled and a curvilinear environment is encouraged.

Finally, the Germans use CCR to describe overall roadway characteristics and to prevent abrupt (and unsafe) transitions in operating speeds between long homogeneous sections of roadways. As previously mentioned, CCR is defined as the absolute sum of the angular changes in the horizontal alignment divided by the length of the highway section.

For a roadway section without transition curves, CCR may be expressed by the following formula (in metric units):

$$CCR = [\Sigma \mid L_i/R_i \mid (63.7)]/L \quad \text{(gon/km)} \qquad (1)$$

or in imperial units:

$$CCR = [\Sigma \mid L_i/R_i \mid (57.3)(2{,}640)]/L \quad \text{(degrees/half-mile)} \qquad (2)$$

where

L_i = length of curve i (ft),
R_i = radius of curve i (ft), and
L = total length of section (ft).

(Note: a *gon* is a unit similar to a degree but related to 400 divisions in a circle instead of 360.)

Use of the design parameter CCR is shown in Figure 7, taken from the German design guidelines *(16)*, with an additional scale added for imperial units. The figure shows the relationship between CCR and 85th-percentile speed and is used to predict the operating speed of any given homogeneous road section. These curves are based on regression analysis of data obtained from actual speed measurements conducted in Germany *(7, 23)*. The allowable speed change between any two consecutive homogeneous road sections is ~6 mph (10 km/hr), and this criterion is used to maintain consistency in the alignment.

To use this method, the first step is to divide the roadway being examined into subsections that have homogeneous alignments. The best way to do this is to construct a cumulative plot of the absolute sum of curvature (1/*R*) versus length for the entire section.

Figure 8 gives this plot for the example alignment in Figure 1. The dotted line represents the theoretical course of the curve if each subsection were perfectly homogeneous, that is, if each curve had exactly the same length and radius and there were no tangents present within the subsection. By comparing the theoretical curve with the actual curve, it is obvious that the road section should be divided into three subsections with nearly homogeneous horizontal alignments: *AF*, *FG*, and *GL*.

Once this has been done, the next step is to calculate the CCR of each subsection by using Equation 2:

Subsection *AF*:

$$CCR = \{[(430/500) + (570/573) + (490/500)] \div (430 + 65 + 570 + 145 + 490)\} (57.3) (2{,}640) = 252.2 \text{ degrees/half-mile}$$

Subsection *FG*:

$$CCR = [(1{,}000/\infty)/1{,}000] (57.3) (2{,}640) = 0.0 \text{ degree/half-mile}$$

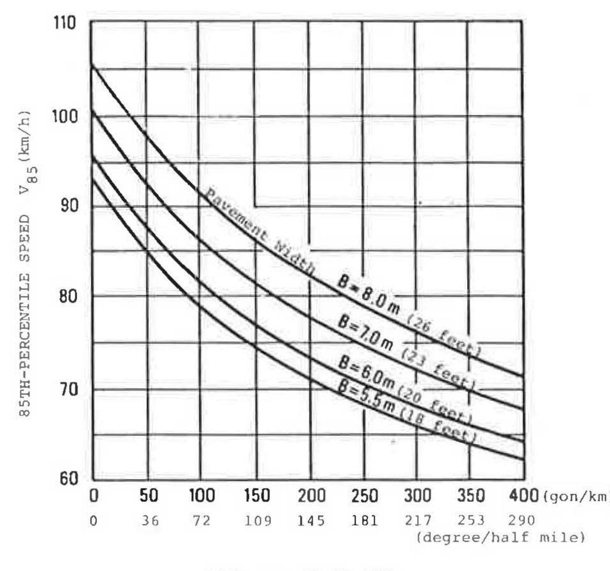

FIGURE 7 Relationship between CCR and 85th-percentile speed in the German guidelines *(16)*.

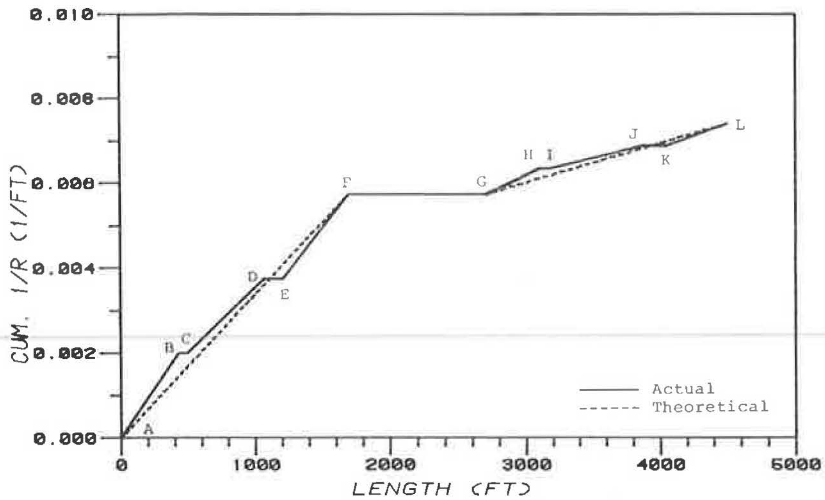

FIGURE 8 Determination of subsections with homogeneous horizontal alignments for roadway section of Figure 1.

Subsection *GL*:

$$CCR = \{[(400/1{,}637) + (690/1{,}910) + (450/1{,}910)] \div (400 + 80 + 690 + 180 + 450)\}\ (57.3)\ (2{,}640) = 70.7 \text{ degrees/half-mile}$$

Using these values and Figure 7 for 10-ft lanes (pavement width 20 ft), the 85th-percentile speed of each subsection may be determined:

- Subsection *AF*: V_{85} = 66 km/hr × 0.62 = 41 mph,
- Subsection *FG*: V_{85} = 96 km/hr × 0.62 = 60 mph,
- Subsection *GL*: V_{85} = 82 km/hr × 0.62 = 51 mph.

These values are valid for both directions of travel, because Figure 7 is based on speed measurements for both directions of travel.

These results indicate that the expected change in operating speed between subsections *FG* and *GL* would be approximately 9 mph, which is slightly over the very strict 6-mph German speed-change limit between successive subsections. Thus there may be a problem with the transition between these two subsections, but it is probably not very serious, especially considering that this value is within the limits recommended by the Leisch and the Swiss methods.

However, between subsections *FG* and *AF* the expected speed difference is approximately 19 mph, which is more than three times the limiting value used in Germany. This finding suggests that a severe inconsistency exists between these subsections. The transition should be investigated more thoroughly, especially considering the accident history of the section, to determine whether any action such as horizontal redesign may be warranted.

The curves in Figure 7 indicate clearly that the pavement width (lane width) of the roadway has an important effect on the operating speed. This example was conducted for a lane width of 10 ft (pavement width 20 ft); obviously different speeds must be expected for different lane widths. It should be noted here that in this comparison the German procedure is the only one that has the effect of lane width built in; the Leisch and the Swiss methods make no provisions for the effect of pavement width on operating speed.

RESULTS AND CONCLUSIONS

The various operating-speed and speed-change values obtained by using each of the three methods are summarized in Tables 3 and 4. Although the differences may be quite substantial at times, the basic conclusions that may be drawn about the investigated road section are the same with each method: the critical speed changes occur before point *A* for west–east travel

TABLE 4 OPERATING-SPEED CHANGES AT THE CRITICAL LOCATIONS OF THE ROAD SECTION IN FIGURE 1

Method	Speed Changes Prior to Point A (West-East)	Speed Changes Prior to Point F (East-West)	Recommended Speed Changes
Leisch	23 mph	22 mph	10 mph
Swiss	19 mph	19 mph	~12 mph
German	19 mph	19 mph	~6 mph

and before point F for east–west travel. Furthermore, it should be emphasized that all three methods produce critical changes in operating speeds larger than any of the maximum allowable speed changes recommended by the different procedures for different countries and continents. These critical values are relatively the same in all three cases, ranging from 19 mph (Swiss and German methods) to 23 mph (Leisch method) (Table 4). If the Leisch method were adapted to the new AASHTO policy on geometric design (Green Book) *(22)*, the critical changes in operating speeds for all three methods would be about the same.

The German CCR method produces the same basic results as those obtained by using speed-profile methods, and it has several advantages over these graphical techniques. The CCR method is based solely on speed measurements and thus reflects the actual driving behavior of motorists, whereas the speed profiles are based largely on theoretical considerations. Also, in their current form the speed-profile techniques have made no provision for the effect of lane width on operating speed.

It would appear that the CCR method would be the most convenient to use in the process of locating inconsistencies in horizontal alignment. It can be easily adapted to the American design system and provides a means of efficiently identifying changes in the operating speed along a highway. It can also be used in connection with RRR projects to locate inconsistencies in alignment transitions and to determine whether a proposed improvement will cause the new roadway section to be designed to a higher standard that is inconsistent with preceding or succeeding highway sections.

The need for a method for achieving consistency in highway operation is emphasized by several findings of an in-depth study team sponsored by the International Road Federation, who surveyed current geometric and pavement design practices in several European countries *(10)*:

- The countries visited place much greater emphasis on achieving consistency among design elements than is called for in U.S. practice.
- In most cases the effect of individual design elements on operating speed is the mechanism for determining design consistency.
- The use of design speed as a concept to be applied to individual elements appears to be diminishing in favor of operating-speed parameters.

Thus it appears that U.S. designers could improve the quality of their design by employing some rational process for predicting the effect of geometry on operating-speed profiles. Adjusting the designs to ensure smoother operating-speed profiles would appear to provide a safety benefit without major cost. In joining their German and Swiss counterparts, U.S. designers could maximize the effectiveness of the significant annual expenditures currently being invested in the U.S. two-lane rural road system through the RRR program. At the very least the procedures outlined in this paper could be used by agencies to identify problem locations and perhaps avoid RRR "improvements" that encourage higher operating speeds and in doing so create a more hazardous environment.

ACKNOWLEDGMENT

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Simulation of Truck Turns with a Computer Model

KENNETH T. FONG AND D. CHARLES CHENU

Recent federal legislation allowing the use of longer and wider trucks will have a significant impact on California's existing roadway system. Many freeway ramps, for example, were designed over a decade ago to accommodate only the largest trucks legally in use at that time. Some of the larger trucks legalized by the new legislation are expected to encounter problems maneuvering through these interchanges. Local governments are also concerned because urban intersections designed many years ago simply cannot accommodate the offtracking of the new larger trucks. To assess the ability of the larger trucks to operate on California's existing roadway system, their offtracking characteristics must be carefully evaluated. In the past, engineers at the California Department of Transportation traditionally used a graphic instrument known as the Tractrix Integrator for simulating truck turns supplemented with a mathematical calculation of the maximum amount of offtracking. A computer model developed for analyzing and evaluating truck offtracking is described. Offtracking results from the computer simulation model are first compared with results derived from the Tractrix Integrator, field observations, and mathematical formulas. The computer model is then used to analyze the offtracking characteristics for several of the new, longer trucks. Finally, applications of the computer model to evaluate some special offtracking situations or problems are discussed.

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The 1982 Surface Transportation Assistance Act (STAA) allowed wider and longer trucks on the Interstate system and portions of the primary system. In 1983 California enacted conforming legislation (Assembly Bill 866). As a result of these legislative changes, a new generation of larger trucks has emerged. The evaluation of the maneuverability of these longer, wider trucks and their ability to operate safely on the roadway system is of prime importance.

PREVIOUS METHODS

Offtracking may be described as "the amount of variation between the path traversed by a following wheel as compared to the path of the preceding wheel" *(1)*. In this paper the center of the axles, rather than the wheels, is used as the reference point for measuring offtracking. Offtracking and related terms are shown in Figure 1.

In California, two methods—the Tractrix Integrator and mathematical formulas—have been used to analyze and evaluate offtracking. The Tractrix Integrator was used to produce

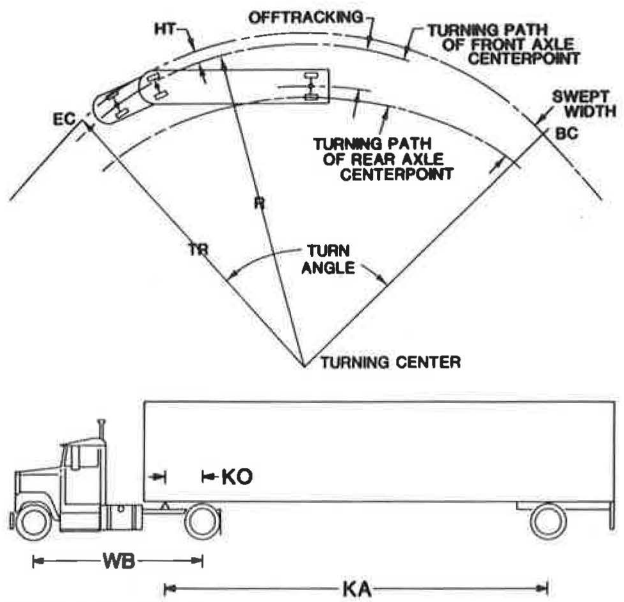

FIGURE 1 Vehicle and offtracking geometries.

offtracking traces and truck turn templates. Mathematical formulas—were used to estimate directly the maximum amount of offtracking.

Tractrix Integrator

Traditionally, highway engineers at the California Department of Transportation (Caltrans) have used a graphic instrument, the Tractrix Integrator, for simulating truck turns (Figure 2). This instrument produces traces of a truck's path that allow the measurement of the amount of offtracking. Thus, one of its main features is that it provides an immediate plot of the truck's path. It is especially well suited for many roadway design situations. Nevertheless, the Tractrix Integrator has several disadvantages. Among them are the following:

- The scale bar cannot be adjusted to accommodate values of less than about 5 ft. Thus, for example, the kingpin is generally assumed to be located directly over the center of the rear tractor axles, and rear overhangs are generally ignored.
- Its use is slow and tedious. To obtain the offtracking path of the first unit of a combination, the pointer of the scale bar first is manually moved carefully along a curve representing the path followed by the center of the front steering axle. Subsequent passes for each unit must be made in order to obtain the path of the center of the rear axle of the rear unit, the pointer in each case following the trace of the previous unit.
- The Tractrix Integrator traces only centerline paths. Consequently, special points of interest (e.g., outside wheels, corners of long rear overhangs, and wide loads) cannot be obtained directly. Artificial lines representing paths of the user-specified point and track widths of the outside front wheel and inside rear wheel, for example, must be manually added to the curves produced from the Tractrix Integrator.
- The Tractrix Integrator used by Caltrans has a bias, probably caused by inexact machining or excessive wear, that causes slightly greater offtracking for right turns than for left turns. To compensate for this bias it is necessary to average the right- and left-turn offtracking of each unit.

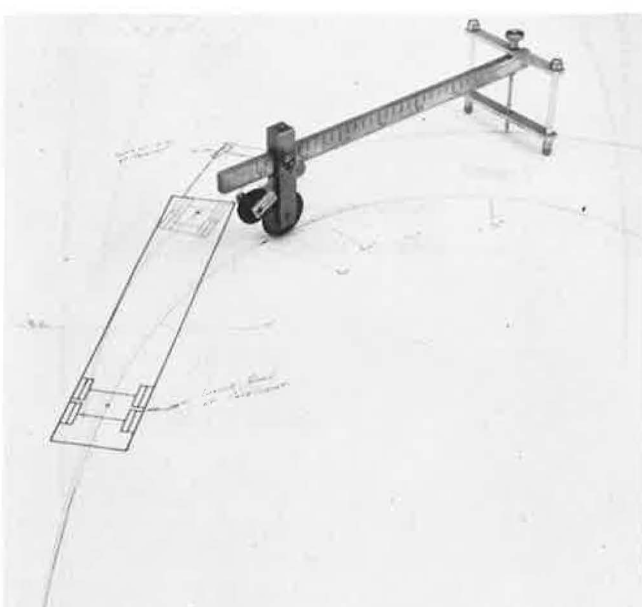

FIGURE 2 Tractrix Integrator.

Mathematical Formulas

Mathematical formulas for estimating maximum truck offtracking were developed by the Society of Automotive Engineers (SAE) in the 1960s. Because these formulas were often very complex and unwieldy, the Western Highway Institute (WHI) in the late 1960s developed simpler but similar equations. The SAE and WHI formulas are widely used by highway engineers to calculate the maximum offtracking expected of a vehicle combination for a curve of a given radius. Although these formulas are widely used, they also are not without shortcomings. They cannot, for example, determine the shape of the spiral path, the amount of offtracking at any point, the location along the path at which maximum offtracking occurs, or whether the maximum value calculated will be reached for a particular curve. Because the location of the maximum offtracking cannot be determined, these formulas are inappropriate in situations where a vehicle pulls out of the turn before the maximum is attained. Both formulas also become indeterminate if the rearmost axle tracks to the inside of the center of the curve, such as on short-radius curves.

A NEW METHOD: COMPUTER MODEL

Anticipating that computer models could provide faster and better solutions to truck offtracking problems, Caltrans started to develop an offtracking model. A literature search indicated a similar project at the University of Michigan. Through contact with FHWA, it was learned that the University of Michigan Transportation Research Institute (UMTRI), as part of a contract with FHWA, had developed a vehicle offtracking computer model, one that in fact simulated the action of the Tractrix Integrator. A detailed description of the nature of the model is presented in the UMTRI report *(2)*.

UMTRI Model

The UMTRI computer model for offtracking simulation is written for the Apple Personal Computer. It is menu driven and easy to use. The program performs the vehicle offtracking simulation by using path and vehicle information supplied by the user and plots selected paths after the simulation. The size of the plot, however, limited by the desktop Apple X-Y Plotter, is relatively small. Also, given a multiunit vehicle or a long path to follow or both, the program will often run out of floppy disk space for storage of the simulation results.

Caltrans Version

Because of the inherent size and capacity limitations of personal computers, Caltrans decided to adapt the simulation portion of the UMTRI model for implementation on the state's IBM mainframe computer and to enhance the program to better meet Caltrans needs. One model has been developed for simple circular curves and a second one for complex compound curves. Three versions (Calcomp drum, Zeta drum, and Xynetics flat-bed plotters) are available for each model. The plots (optional) are the same except that the Calcomp and Zeta plotters have a maximum paper width of 34 in. and virtually unlimited length, whereas the Xynetics plotter has a maximum plotting area of 42 × 88 in. In addition, the Caltrans computer model produces several printed reports.

Supported by IBM's MVS Operating System, the Caltrans offtracking model runs extremely quickly. Only a fraction of a second in computer-processing-unit time is required to execute a simulation run. If the job is submitted via a time-sharing system from a video display terminal, the user can preview the printed output in a matter of minutes. Processing costs vary from less than \$0.25 to about \$3.00 if a plot tape is generated.

COMPARISON OF COMPUTER MODEL RESULTS

A 48-ft test semitrailer (tractor-semitrailer combination) was used in the comparison of the results from the Caltrans computer model with centerline traces obtained by using the Tractrix Integrator, swept widths observed from an actual field test, and maximum offtracking values calculated from mathematical formulas. Figure 3 shows the 48-ft test semitrailer configuration and key dimensions.

Caltrans Model Versus Tractrix Integrator

The 48-ft test semitrailer was simulated and centerline axle traces were plotted for a 180-degree turn with a 50-ft radius at a scale of 1 in. = 5 ft. Tractrix centerline traces (for right turns and mirror images of left turns) were superimposed on the

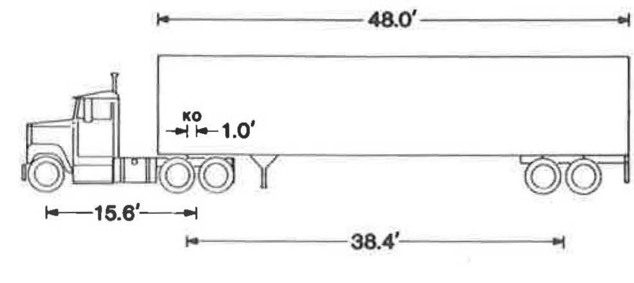

FIGURE 3 Test semitrailer with 48-ft trailer.

computer plot. The two sets of traces (computer generated and average Tractrix) were almost identical (Figure 4).

The computer plots are in fact better than the Tractrix drawings. As mentioned earlier, the Tractrix Integrator produces traces only for vehicle centerlines and requires rear-wheel paths, for example, to be manually constructed. The computer model, on the other hand, can plot any user-specified vehicle reference points. In addition, as also mentioned earlier, the California Tractrix Integrator has an offtracking bias that is not in the computer model.

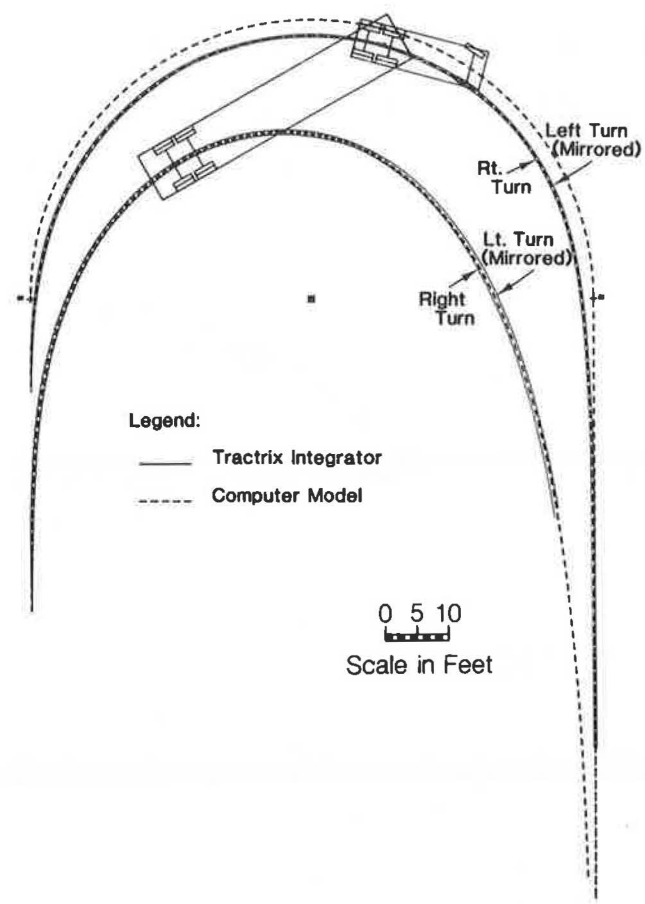

FIGURE 4 Centerline traces.

TABLE 1 RESULTS FROM FIELD TESTS COMPARED WITH COMPUTER MODEL FOR SWEPT WIDTH (FT): 48-FT TEST SEMITRAILER NEGOTIATING A 180-DEGREE 50-FT-RADIUS CURVE

	Angle Ahead of BC (Degrees)						
	0	30	60	90	120	150	180
Field Test	13.5	20.1	23.8	26.2	27.9	27.8	24.0
Computer Model	13.7	20.2	24.1	26.7	28.3	28.1	23.9
Difference	0.2	0.1	0.3	0.5	0.4	0.3	-0.1
% Error	1.5	0.5	1.3	1.9	1.4	1.1	-0.4

Caltrans Model Versus Actual Field Test

In 1984 Caltrans conducted an actual field test of the 48-ft test semitrailer *(3)*. It was driven around a 50-ft-radius, 180-degree curve in a parking area, and the amount of swept distance was recorded at 30-degree increments from the beginning of the curve (BC). Field test results are compared with those from the computer model in Table 1. It may be seen that the results are close. The maximum difference is only 0.5 ft, an error of less than 2 percent.

Caltrans Model Versus Mathematical Formulas

A summary of the offtracking results from the computer model for the 48-ft test semitrailer negotiating a 50-ft radius curve is given in Table 2, which shows the amount of offtracking (in feet) at the beginning and the end of a curve, the maximum offtracking value reached, and where along the path the maximum occurred.

TABLE 2 OFFTRACKING RESULTS BY DEGREE OF TURN: 48-FT TEST SEMITRAILER NEGOTIATING A 50-FT-RADIUS CURVE

Degree	Offtracking (in Feet)			Location of
of Turn	BC	EC	Maximum	MOT (deg)
30	5.0	6.2	6.8	19
60	6.1	10.0	11.6	39
90	6.1	12.5	15.2	61
120	6.1	14.2	17.7	85
150	6.1	15.4	19.6	109
180	6.1	16.3	21.0	133
210	6.1	16.9	22.1	159
240	6.1	17.4	22.8	186
270	6.1	17.7	23.4	213
300	6.1	18.0	23.9	241
330	6.1	18.2	24.2	269
360	6.1	18.3	24.4	297

For comparison, the maximum offtracking values computed from the SAE and WHI formulas *(1, 4)* for the 48-ft test semitrailer negotiating a circular curve with a 50-ft radius are as follows:

SAE formula:

$$\begin{aligned} OT &= \{WB^2 + [(TR^2 - WB^2)^{1/2} - HT]^2\}^{1/2} \\ &\quad - \{KO^2 + [(TR^2 - WB^2)^{1/2} - HT]^2 - KA^2\}^{1/2} \\ &= \{15.6^2 + [(50^2 - 15.6^2)^{1/2} - 33]^2\}^{1/2} \\ &\quad - \{1^2 + [(50^2 - 15.6^2)^{1/2} - 3.33]^2 - 38.4^2\}^{1/2} \\ &= 25.0 \text{ ft} \end{aligned} \tag{1}$$

WHI formula:

$$\begin{aligned} MOT &= R - (R^2 - \Sigma L^2)^{1/2} \\ &= 46.67 - [46.67^2 - (15.6^2 - 1^2 + 38.4^2)]^{1/2} \\ &= 25.2 \text{ ft} \end{aligned} \tag{2}$$

where

- OT or MOT = offtracking (maximum or steady-state),
- WB = wheelbase of tractor,
- TR = turning radius of outside front tire,
- HT = half of front-axle track width,
- KO = kingpin offset (fifth wheel) of tractor,
- KA = kingpin to centerline of rear axle group of semitrailer,
- R = $TR - HT$ = radius followed by front-axle center, and
- ΣL^2 = $WB^2 - KO^2 + KA^2$ = sum of square of component lengths between axle spacings.

As mentioned earlier, the mathematical formulas can only give the maximum offtracking value expected; they cannot tell where the maximum will occur. From the computer model, the maximum offtracking attained by the 48-ft test semitrailer making a 90-degree turn is just 15.2 ft, or about 10 ft less than

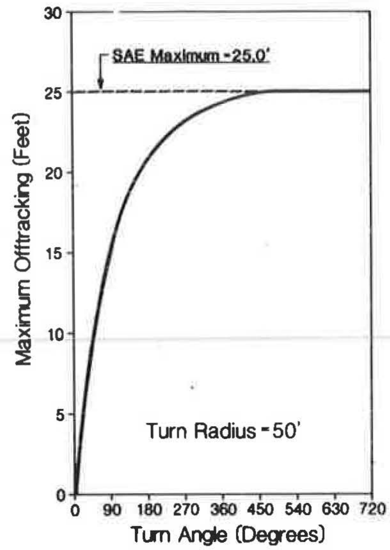

FIGURE 5 Maximum offtracking by turn angle for 48-ft test semitrailer.

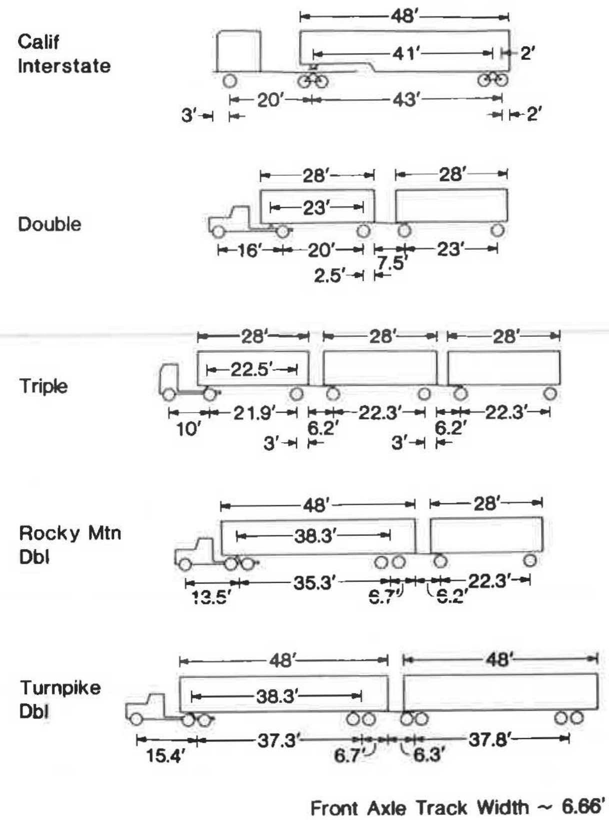

FIGURE 6 Truck configurations.

the mathematical maximum. On a 180-degree turn, the maximum offtracking value from the computer model is 21.0 ft, or about 4 ft less than the mathematical maximum. And on a 270-degree turn, the maximum is 23.4 ft. This is almost 2.0 ft less than the expected maximum calculated from the mathematical formulas.

Table 2 shows clearly that as the degree of turn increases, the maximum offtracking value also increases, but at a progressively slower rate. It also suggests that if given enough angular rotation, the maximum offtracking value from the computer model will eventually reach the mathematical maximum. This is verified with additional results developed from the computer model. The relationship between maximum offtracking and degree of turn is shown in Figure 5.

OFFTRACKING RESULTS OF LONGER TRUCKS

The computer model was next applied to analyze the offtracking characteristics for the post–1982 STAA California Interstate design vehicle and several of the longer vehicle combinations. These vehicle configurations are shown in Figure 6.

Since enactment of the 1982 STAA, Caltrans designers have been using two design vehicles. The Interstate design vehicle is for use on the Interstate system, non-Interstate freeways, and some conventional highways. The non-Interstate design vehicle (not shown) is used for the remainder of the California highway system.

The first two vehicles in Figure 6, the California Interstate design vehicle and a twin trailer truck with 28-ft twin trailers, are now legal in California. The last three—a Rocky Mountain double, turnpike double, and a triple trailer truck with three 28-ft trailers—are not currently allowed. This latter group of longer combination vehicles is under study as prompted by Section 138/415 of the 1982 STAA.

The computer model was used to simulate all five vehicle types negotiating simple circular curves of various radii and central angles. The maximum offtracking values for these vehicles on a 180-degree turn with radii of 60, 100, and 150 ft are summarized in Table 3. For comparison, the maximum offtracking values calculated from the SAE and WHI equations are also given.

The following observations may be made from Table 3:

- The maximum offtracking values calculated by using the SAE and WHI formulas differ only slightly and in most cases are identical.
- The maximum offtracking values from the computer model and mathematical formulas are the same for trucks making the longer-radius turns (e.g., ~100 ft). For the shorter-radius turns, the difference in offtracking may be substantial. For example, on a 180-degree turn at a radius of 60 ft, the difference in offtracking is about 14 ft for the turnpike double. It should be pointed out that on an actual field test, the maximum offtracking observed for the turnpike double negotiating a 180-degree turn on a 60-ft-radius was 32.7 ft, with the measured maximum located about 120 degrees from the beginning of the curve *(3)*. This value also confirms the maximum offtracking value from the computer model.
- Amount of offtracking varies inversely with the radius of turn. The shorter the radius, the greater the amount of offtracking. Offtracking is very sensitive at the shorter-radius turns and becomes relatively inelastic for the wider-radius turns.

As indicated earlier, the mathematical equations provide only the theoretical or steady-state maximum value. The simulation

TABLE 3 MAXIMUM OFFTRACKING VALUES (FT) FOR TRUCKS MAKING A 180-DEGREE TURN

	Turn	Maximum Offtracking (in Feet)			
	Radius	Computer Model		Math Formulae	
Vehicle Description	(ft)	Max	@Deg	SAE	WHI
California	60	20.8	134	22.9	23.0
Interstate	100	11.4	147	11.4	11.4
Design Vehicle	150	7.3	157	7.3	7.3
Double 28'	60	13.3	129	13.5	13.6
all state hwys	100	7.3	144	7.3	7.3
after 1983 AB 866	150	4.7	157	4.7	4.7
Triple 28'	60	16.7	124	17.3	17.3
under Federal study	100	9.0	138	9.0	9.0
per 1982 STAA	150	5.8	154	5.8	5.8
Rocky Mountain double	60	21.7	128	23.7	23.8
under Federal study	100	11.7	144	11.7	11.7
per 1982 STAA	150	7.5	156	7.5	7.5
Turnpike double	60	33.0	123	46.9	47.5
under Federal study	100	17.7	136	17.8	17.8
per 1982 STAA	150	11.1	149	11.1	11.1

model, on the other hand, determines the maximum amount of offtracking for a specific degree of turn. The two values (from the equation and simulation model) will be the same only if the degree of turn is sufficient to allow the vehicle to reach its steady-state condition. It is often necessary for a vehicle to travel more than 180 degrees (particularly on short-radius curves) to reach its steady-state condition. In addition, the SAE and WHI formulas cannot determine the shape of the curve going to and from the point of the maximum offtracking or where the maximum value will occur. An important feature of the computer model is that it can keep track of where the truck is at any given instant. The amount of offtracking and its location are routinely reported as the truck moves along its prescribed path. This can be very helpful in the analysis and evaluation of offtracking problems.

The results from the computer model are used in Figure 7 to show the maximum offtracking of the California Interstate design vehicle by turn angle for different turn radii. Similar graphs may be made for the other trucks as well, but only one is presented to illustrate the relationship that offtracking for a particular vehicle configuration is a function of both the turn radius and the turn angle.

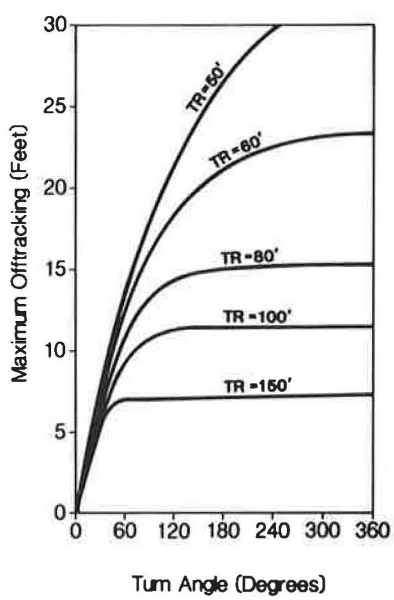

FIGURE 7 Maximum offtracking by turn angle and radius for California Interstate design vehicle.

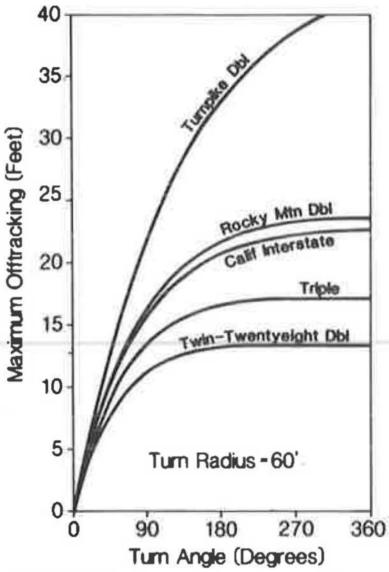

FIGURE 8 Maximum offtracking by turn angle for selected trucks.

Figure 8 is similar to Figure 7, but instead of various turn radii and central angles, offtracking for various vehicle combinations negotiating a common radius curve through various turning angles is shown. From Figure 8 it may be seen that

- The turnpike double offtracks the greatest amount, and the twin with 28-ft trailers offtracks the least. On a 180-degree turn with a 60-ft radius, the turnpike double offtracks almost 20 ft more than the twin.
- The Rocky Mountain double and the California Interstate design vehicle have similar offtracking characteristics, but the Rocky Mountain double offtracks slightly more than the California Interstate design vehicle.
- The amount of offtracking for the triple with 28-ft trailers falls somewhat between that of the California Interstate design vehicle and the twin.
- None of the vehicle combinations would be able to negotiate a 60-ft-radius right-angle turn, such as that found at urban intersections, without tracking outside of a normal 12-ft lane. For example, the twin would require 11.4 ft of offtracking plus 8.5 ft of track width, or about 20 ft of swept width.

SPECIAL OFFTRACKING STUDIES

Applications of the computer model to evaluate special offtracking situations or problems are discussed in the following sections. Some of these special offtracking studies include

- Backtracking and pivoting of rear trailer wheels,
- Effect of kingpin placement,
- Boom carriers, and
- Compound curves.

Backtracking and Pivoting

Backtracking and pivoting is the stopping and backing up, with or without pivoting, of the rear trailer tires while the tractor follows a specified uniform path. This occurs when long vehicle combinations negotiate curves with a very short radius and large central angle. Caltrans has recently begun using the computer simulation model to investigate this problem.

As previously mentioned, mathematical formulas cannot be used in short-radius turns where the rearmost axle tracks to the inside of the radius center. The computer model overcomes this limitation quite easily. To demonstrate this ability, Figure 9 shows a computer plot of the California Interstate design vehicle negotiating a 180-degree turn on a 25-ft-radius curve. The backtracking and pivoting of the semitrailer behind the curve center is readily identified. It should be pointed out, however, that the computer model does not calculate the minimum turn radius, that is, the sharpest curve that can be made by a truck. It is up to the user to determine whether any such short-radius turn is actually possible for a particular type of truck.

Kingpin Placement

The computer model was used to investigate the effect of the placement of the fifth wheel (kingpin offset) on the amount of offtracking.

The offtracking results from the computer model for three types of trucks negotiating a 180-degree turn at radii of 60, 100,

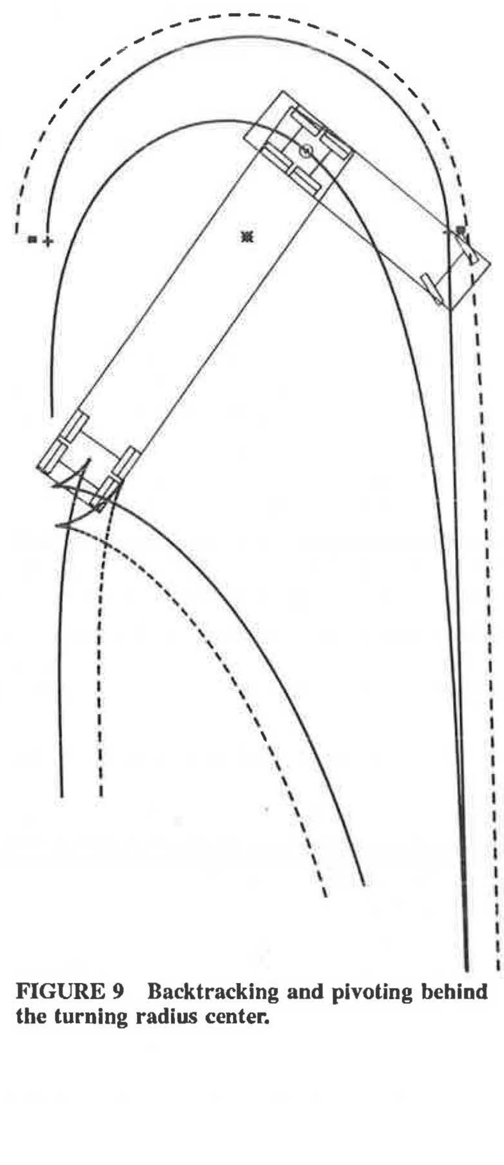

FIGURE 9 Backtracking and pivoting behind the turning radius center.

TABLE 4 MAXIMUM OFFTRACKING (FT) FOR CENTRAL ANGLE OF 180 DEGREES

	Kingpin Offset (Feet)						Max.
	0	±1.0	±2.0	±3.0	±4.0	±5.0	Diff
TR = 60'							
Calif Interstate	20.84	20.83	20.80	20.74	20.67	20.57	.27
Rocky Mtn Dbl	21.81	21.80	21.76	21.71	21.63	21.53	.28
Triple	16.70	16.69	16.65	16.60	16.53	16.42	.28
TR = 100'							
Calif Interstate	11.40	11.40	11.38	11.35	11.31	11.26	.14
Rocky Mtn Dbl	11.77	11.76	11.75	11.72	11.68	11.63	.14
Triple	9.00	9.00	8.98	8.95	8.91	8.86	.14
TR = 150'							
Calif Interstate	7.28	7.28	7.27	7.25	7.22	7.19	.09
Rocky Mtn Dbl	7.50	7.50	7.49	7.47	7.44	7.41	.09
Triple	5.78	5.78	5.77	5.75	5.73	5.69	.09

and 150 ft are given in Table 4. Different placement of the kingpin on the tractor was assumed. These results (and those for a 90-degree central angle, which is not shown) reaffirm the correctness of the mathematical formulas (Equations 1 and 2) and indicate that

- The maximum offtracking occurs when the kingpin is located directly over the rear tractor axle or axles;
- Offtracking decreases as the kingpin is moved away (either ahead of or behind) the rear tractor axle or axles;
- Corresponding kingpin locations ahead of and behind the rear tractor axle or axles cause the same amount of offtracking; and
- The effect of the kingpin placement on offtracking is negligible. Even when the kingpin is offset 5 ft, the maximum offtracking on a 180-degree turn with a 60-ft radius is only 0.28 ft (about 3.5 in.).

Boom Carriers

The use of the computer simulation model to track points selected by the user is shown in Figure 10. The original plot was at a scale of 1 in. = 5 ft. It has been reduced for this paper.

Simulations of various boom lengths for both front and rear boom carriers have been made. In this example an unsupported 32.5-ft front boom carrier is shown on a 90-degree 60-ft-radius curve. Points of interest that were plotted include the corners of the boom overhang and the right carrier overhang. The boom overhang is particularly important because it exhibits consider-

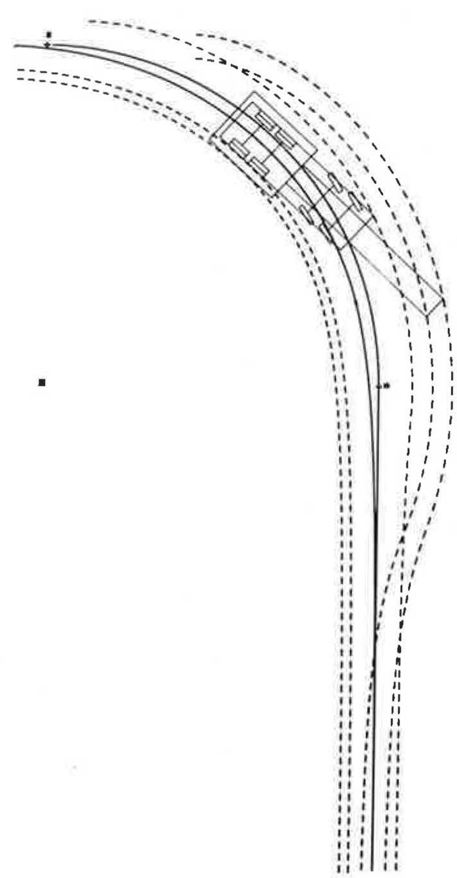

FIGURE 10 Path traces of special points for unsupported front boom carrier.

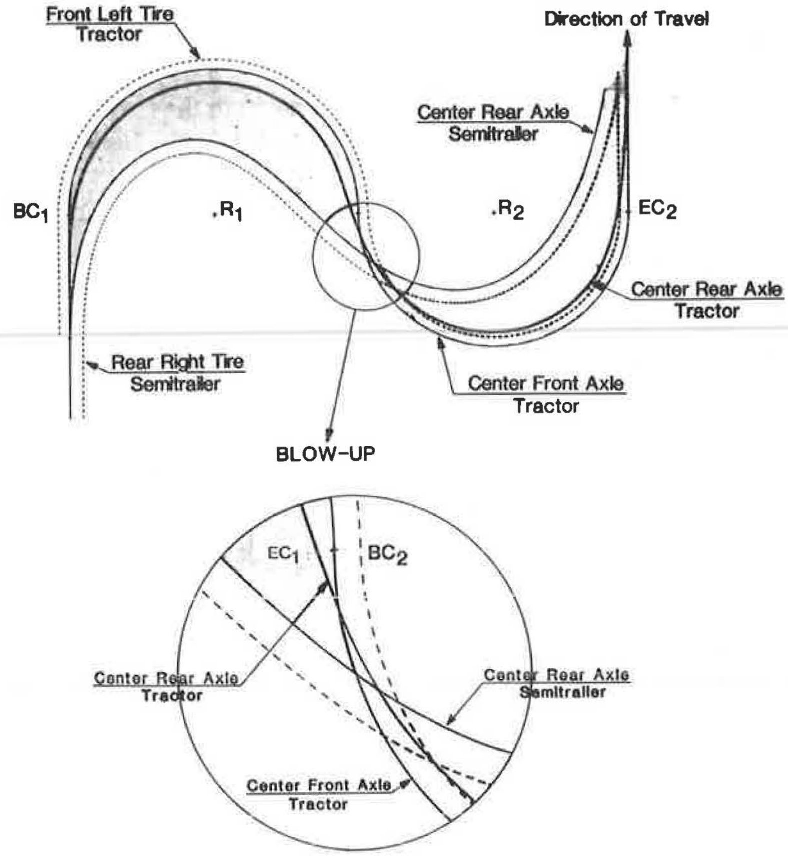

FIGURE 11 Tractor-semitrailer negotiating an S-curve.

able negative offtracking (often overlooked when offtracking is analyzed), which extends well past the end of the curve (EC). Rear boom carriers, on the other hand, generally have an outswing that starts before the beginning of the curve (BC).

Currently only plots of the special point traces may be obtained from the computer simulation model. Calculation of the amount of offtracking for user-specified points is not available at this time. These values will be added in the future to an updated version of the program.

Compound Curves

Offtracking has been defined in several ways, the simplest being "the additional width (over and above the truck width) required by a vehicle when making a turn" *(3)*. Because of the complex paths followed by vehicles negotiating compound curves and because offtracking continues after a vehicle has completed its turn, a definition that is both more flexible and precise is needed. It appears satisfactory (provided the rear axle does not swing inside of the curve center) to define the offtracking as the amount of variation between the path traversed by a point on the steering axle and the path of the corresponding point on the subsequent axle (or axles if the steering axle path is not outermost) that has the greatest variation, when measured normal to the path of the front axle. The formulation of a more universal definition that covers all situations is needed.

In Figure 11 a tractor-semitrailer is shown negotiating a reverse curve or S-curve. In this plot the complex paths and offtracking that result when a vehicle negotiates a sequence of curves of different radii with changes in the direction of travel and the difficulty in defining (much less measuring) the amount of offtracking may be seen. At this time the offtracking (and swept-width) values are not calculated; instead, only the traces are drawn.

SUMMARY

Recent legislation allowing the use of wider and longer trucks will require careful evaluation of the maneuverability of these vehicles. With the rapid increase in the use of computers, highway engineers are turning to computer models for faster and better answers to offtracking problems for a range of new and proposed configurations. Caltrans has recently implemented a computer model that has proved superior to methods used in the past. The computer model has provided new insights into many offtracking problems. It is extremely fast, efficient, and economical to use. Offtracking simulation models are expected to evolve rapidly in the 1980s. This exciting new computerized method will be the chief analytical tool for solving offtracking problems in the future.

ACKNOWLEDGMENTS

The contributions of Michael Sayers of the University of Michigan Transportation Research Institute, who developed the initial vehicle offtracking computer simulation model, and Michael Freitas of FHWA, who provided the UMTRI Apple computer program, are acknowledged. The research presented in this paper was conducted in cooperation with FHWA.

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Two-Lane Traffic Simulation: A Field Evaluation of Roadsim

JUAN M. MORALES AND JEFFREY F. PANIATI

Roadsim is a traffic simulation model for two-lane rural roads developed in 1980 by FHWA. In the subject study the accuracy of the model was evaluated by comparing its results with observed traffic behavior. The field data were collected on a two-lane rural road in Loudoun County, Virginia. Statistical analyses were performed to compare the measures of effectiveness (MOEs) observed in the field with those obtained from the simulation. The selected MOEs included mean vehicle speed, traffic volume, percent of vehicles following, platoon distribution, and average platoon size. Analysis showed that Roadsim's simulation results compared favorably with those observed in the field. Although this study validates Roadsim under a single geometric and traffic condition, results support its potential usefulness to the transportation engineering community. Further validation under a wide range of traffic and geometric conditions, however, is needed. Researchers are encouraged to use Roadsim to further validate its potential and recommend enhancements.

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Traffic simulation, a tool used by traffic engineers in the analysis of roadway capital investment and traffic control management, provides valuable information to decision makers by predicting the likely effects of traffic or geometric changes on a roadway before the changes actually occur. Simulation results may be used to decide whether to proceed with the change, modify it, or abandon it. Simulation may determine the most effective way to spend available funds.

Initially, traffic simulation was directed to the urban scene. Because urban intersection traffic essentially behaves as a multilane queueing system, traffic may be simulated by using techniques developed for operations research. Simulation of freeway ramp traffic required modeling of traffic behavior by using queueing analogies. Freeway simulation studies were the pioneers of traffic simulation as a research tool.

Simulation of rural traffic on two-lane roads developed at a slower pace because the two-lane flow is complicated by platooning and passing decisions and therefore not easily modeled. Also, the low volumes on rural two-lane roads usually do not make simulation cost-effective. In addition, two-lane traffic simulation requires numerous computations, which require considerable computer time and memory, particularly for microscopic models. To date, most of the two-lane simulation models are microscopic. These models simulate and trace individual vehicles and are more accurate and realistic than macroscopic models, which simulate traffic using aggregate variables such as traffic volume and average speed.

Simulation models for two-lane roads have evolved over the past two decades. Most of the early attempts contributed little to the study of two-lane flow at a practical level. However, those attempts were stepping stones for other sophisticated simulation models currently available.

The ability of Roadsim, a traffic simulation model for two-lane rural roads that was developed in 1980 for FHWA, to replicate traffic operations observed on an existing two-lane rural road is evaluated. Field data were collected on a two-lane rural road in Loudoun County, Virginia. Statistical analyses performed to compare the measures of effectiveness (MOEs) observed in the field with those obtained from the simulation show that Roadsim's simulation results compare favorably with those observed in the field. Results support its potential usefulness to the transportation engineering community after the model has been further validated under a range of traffic and geometric conditions.

EVOLUTION OF ROADSIM

Roadsim, the latest product of the evolutionary process of two-lane simulation model development, is not a new model with new methodology and logic but rather a reprogrammed version of an earlier model (called TWOWAF) with modified routines and adaptations from other models *(1)*.

TWOWAF, a microscopic traffic simulation model, was developed in 1978 as part of the National Cooperative Highway Research Program (NCHRP) Project 3–19 *(2)*. The model can move individual vehicles in accordance with several parameters specified by the user. The vehicles are advanced through successive 1-sec intervals, and the roadway geometry, traffic control, driver preferences, vehicle type and performance characteristics, and passing opportunities based on the oncoming traffic are taken into account. Spot data, space data, vehicle interaction data, and the overall traffic data are accumulated and processed. Several statistical summaries are reported.

TWOWAF logic was modified to include logic elements from two other simulation models—INTRAS and SOVT *(3)*. INTRAS, a microscopic freeway simulation model developed in 1976 for FHWA, provided the basic car-following logic to TWOWAF. This logic is based on the premise that a vehicle that is following another will always maintain a space headway relative to its lead vehicle that is linearly proportional to its speed. This premise was much simpler than the one used in TWOWAF and thus easier to calibrate. SOVT, a microscopic two-lane simulation model developed in 1980 at North Carolina State University, provided its vehicle generation logic to TWOWAF. This logic emits vehicles onto the simulated roadway at each end. For low volumes, the Schuhl distribution used in SOVT provides a realistic approximation of vehicles generated. However, for high volumes where traffic density approaches queueing, a shifted exponential headway distribution is used.

The new TWOWAF model was reprogrammed according to FHWA specifications, modified with new input and output subroutines, and renamed Roadsim. Detailed documentation was made available as part of TRAF, an integrated system of simulation models *(1)*. This evolutionary process is shown in Figure 1.

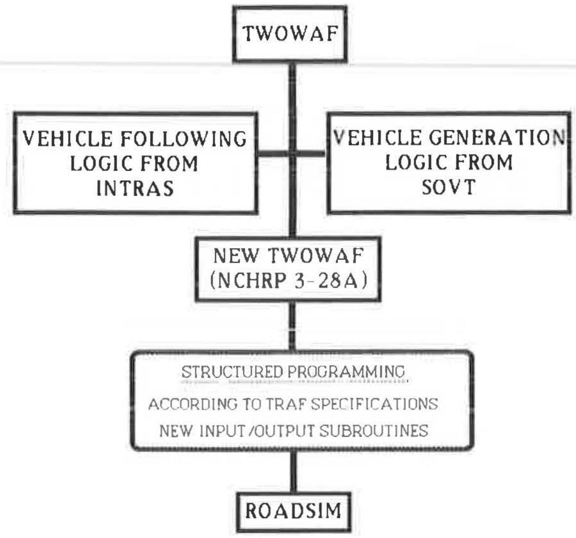

FIGURE 1 Evolution of Roadsim.

MOES GENERATED BY ROADSIM

Roadsim is structured in a link-node format, which requires the simulated roadway to be divided into segments called links. Links are interconnected at points called nodes. It is through the links that the roadway geometrics are specified to the model.

In addition to overall statistics, some of the MOEs generated by Roadsim are reported as link-specific or link- and direction-specific. Link-specific MOEs are generated for each direction of travel. MOEs and their units reported in the cumulative output of Roadsim are given in Table 1.

DATA COLLECTION AND METHODOLOGY

Site

A 4.6-mi (7.4-km) section of US-15 in Loudoun County near Leesburg, Virginia, was chosen as the site for the data collection on the basis of the following geometric and operational factors:

- Significant truck volume,
- Rolling terrain,
- Minimal roadside activities,
- No major intersections,
- Standard roadway features (e.g., signing, shoulder width, sight distance),

TABLE 1 MOEs OF EFFECTIVENESS GENERATED BY ROADSIM

Measure	Units
Link specific and direction specific by vehicle category (automobile, recreational vehicle, truck)	
Travel	Vehicle-miles
	Vehicle-trips
Travel time (ideal, zero traffic, and actual)	Seconds/vehicle
Standard deviation of travel time	Seconds
Delay (geometric, traffic, and total)	Seconds/vehicle
Standard deviation of delay	Seconds
Mean speed, standard deviation of speed, speed extremes	Miles/hour
Passes attempted, completed, and aborted	Number per mile per hour
Link specific	
Distribution of headways	Number in each range, percent of total, cumulative percent
Distribution of speeds	Number in each range, percent of total, cumulative percent
Distribution of platoon sizes	Number in each range, percent of total, cumulative percent

Note: 1 mi = 1.6 km.

- Adequate two-way volume to cause significant platooning and passing opportunities, and
- Attainable free-flow speed of 55 mph (89 km/hr) or faster.

Road geometry data, obtained from construction plans supplied by the Virginia Department of Highways and Transportation (VDHT), included horizontal and vertical alignment (Figure 2). Passing zones and link lengths were measured in the field by using a calibrated fifth wheel. Information on sight distance was computed manually with the following formula:

Maximum passing sight distance = length of passing zone + 1,500 ft (1)

Volume and other traffic characteristics were measured in the field. Route 15 carries a significant truck volume because its weight limits are higher than those of adjacent roadways. Observed traffic volume during most of the daylight hours was between 300 and 400 vehicles/hr (in both directions) with 25 percent trucks. These characteristics were desirable for the study because low volumes create frequent passing opportunities and the high percentage of trucks creates platoons.

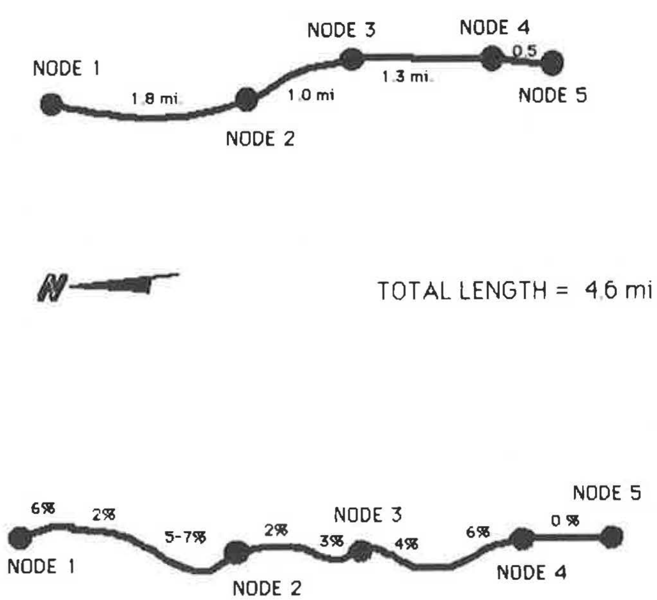

FIGURE 2 Geometric characteristics: ***top*, horizontal alignment; *bottom*, vertical alignment.**

Procedure

Two-way traffic was observed on the selected roadway section, which was divided into four links based on the geometric similarities of the roadway within each link. Data were collected at each node (called stations) using color videotape recording equipment. The recording procedure was chosen for the following reasons:

- Data reliability was high,
- Staff requirements were low (one person per node),
- Manual data logging in the field was not required,
- Vehicles could be tracked without the recording of license plate numbers,
- Data could be easily verified and corrected,
- A permanent record of the data was available for future studies, and
- Equipment cost was low.

Each node required a color videotape recorder, a camera, a power supply, a digital stopwatch, and a tripod. The average setup rental was $100 per day.

Both equipment and attendants were stationed in an unobtrusive location off the roadway. All cameras were positioned at the same angle to obtain similar views of each vehicle and facilitate vehicle tracking from node to node.

Data were collected for three 2-hr periods over 2 days. The video recorders were run in real time for the duration of each period. Digital watches were used to synchronize the cameras. Each camera attendant audibly recorded the time on the recorder every 15 min to provide a time reference during data reduction.

DATA REDUCTION

The videotaped traffic data were manually coded onto data forms. This task required approximately 48 person-hr to reduce each of the three 2-hr data collection periods for all five nodes.

The data obtained from the videotapes were arrival time (to the nearest second), vehicle type (atuomobile, recreational vehicle, single-unit truck, or combination truck), and vehicle description for tracking purposes (e.g., color, make, model).

Although the roadway section selected contained no major intersections, there were several residential driveways and two minor intersections. Eight percent of the observed vehicles did not travel the entire roadway (entry at Node 1 and exit at Node 5) and so were not included in the data analysis. Data, entered into an electronic spreadsheet for compilation, could be corrected and updated. After the data were input, they were checked for errors against the videotapes.

Vehicle data were stored separately for each direction of travel. Vehicles were numbered sequentially on the basis of their arrival order at the entry node. The difference between a vehicle's arrival times at the individual nodes determined its travel time for each link. Speeds were obtained by dividing the length of each link by the travel time. Headway was defined as the difference between the arrival time of a vehicle and the arrival time of the next vehicle.

To complement the spreadsheet, programs were developed to compute platoon sizes and the number of completed passes. Platoon sizes were computed after it had been determined whether a vehicle was a leader or a follower. By definition, a vehicle was said to be following another if its bumper-to-bumper headway was 6 sec or less. This is the same headway used by Roadsim for this purpose. Each platoon consisted of a leader and its followers, if any.

The number of completed passes was determined by comparing the arrival sequence at individual nodes with the sequence at the previous node. Separate data were obtained from the spreadsheet for the four vehicle types for comparison with Roadsim. The data included mean speed, headway, travel time, and number of completed passes. Some data had to be discarded after careful examination; for instance, artificial delays were created because of extremely slow vehicles (tractors) in the traffic stream, and the simulation is unable to represent this. Of the 6 hr of traffic data collected, two segments (one 30-min and one 60-min) were used in the comparative analysis.

Comparison of the two selected periods showed their traffic flow characteristics to be different. Therefore, they were compared separately with the simulation results. The factors considered were variations in traffic volume, vehicle mix, directional split, and platooning because these data have to be input into the model.

The reduced data included statistics for the overall roadway length as well as for individual links and vehicle types. These data, along with the spreadsheet templates (LOTUS 1-2-3, which is IBM compatible) and other programs (IBM BASIC) generated for this study, are available to other researchers through the authors.

THE SIMULATION PROCEDURE

Once the field data were reduced, Roadsim was coded and executed to obtain data for comparison.

Coding Roadsim

To replicate field conditions and simplify coding the model, the following assumptions were made:

- All vehicles fell into one of four possible vehicle types: type 1—automobiles, vans, pickup trucks; type 2—recreational vehicles, horse trailers, tow trucks; type 3—single-unit trucks, school buses, sanitation trucks; or type 4—combination trucks.
- Field maximum passing sight distance, required in the input stream, was determined by adding the length of the passing zones to 1,500 ft (457 m) (VDHT standard minimum), as previously explained.

Several default values contained in the model that were judged adequate and compatible with the field data were used to simplify coding. The data required to run the model and the default values used in this study are given in Table 2.

Coding the required input was tedious because interactive data input procedures were not available. The model is coded by entering data into specific fields of 80-column cards from a mainframe computer terminal. This required constant reference to the User's Guide *(1)* and several runs to correct misplaced data entries. The User's Guide, however, contains a complete error message section that proved to be very useful in completing this task.

Adjusting Roadsim

To simulate the observed field conditions, the model's control input had to be adjusted initially. These adjustments are not to be confused with model calibration, which refers to the fine tuning of empirical coefficients in the actual computer code. The adjustments were made to the control data and not to the Roadsim code. Because of the random nature of traffic behavior, these adjustments were necessary to ensure that the collected field data could be directly compared with the simulation data. Other adjustments made because of the input and output formats of the model are discussed in the following sections.

Model Links Versus Field Links

Because the Roadsim input format allows the user to specify only one horizontal curve, two vertical curves, and three no-passing zones per link, it was necessary to divide the four field links into seven smaller model links.

TABLE 2 REQUIRED DATA AND VALUES USED

Variable	Comment or Value
Free-flow speed	Variable (see text)
Standard deviation	9 percent of free-flow speed
Forward sight distance	1,500 ft
No-passing regions	Variable (three per link maximum)
Link length	Variable (9,999-ft maximum)
Passing sight distance	Variable (three regions per link maximum)
Horizontal curve data	Variable (one curve per link maximum)
Length	
Radius	
Superelevation	
Vertical curve data	Variable (two curves per link maximum)
Length	
Grade	
Vehicle type data	Variable (16 types maximum)
Automobiles	
Length	17 ft[a]
Maximum acceleration	5.5 mph/sec[a]
Maximum speed	75 mph[a]
Maximum entry speed	75 mph[a]
Volume	Variable (vph/direction)
Recreational vehicles	
Length	25 ft[a]
Maximum acceleration	5.9 mph/sec[a]
Maximum speed	65 mph[a]
Maximum entry speed	65 mph[a]
Volume	Variable (vph/direction)
Single-unit trucks	
Length	30 ft
Weight/horsepower (power factor)	72 lb/horsepower
Weight/frontal area (mass to frontal area factor)	158 lb/ft[a]
Elevation factor	1.0
Drag factor	0.96
Maximum entry speed	65 mph
Volume	Variable (vph/direction)
Combination trucks	
Length	65 ft
Weight/horsepower (power factor)	266 lb/horsepower
Weight/frontal area (mass to frontal area factor)	620 lb/ft[a]
Elevation factor	1.0
Drag factor	0.96
Maximum entry speed	65 mph
Maximum acceleration using partial horsepower	81 percent
Maximum 0-grade speed using partial horsepower	90 percent[a]
Pass suppressing influence upstream of curve to right	10 sec[a]
Bias to add to trucks' desired speeds	–1.5 ft/sec[a]
Bias to add to recreation vehicles' desired speeds	–2.2 ft/sec[a]

Note: 1 ft = 0.305 m; 1 lb/horsepower = 0.608 kg/kw; 1 mph/sec = 1.01 km/sec; 1 mph = 1.6 km/hr; 1 lb/ft^2 = 4.88 kg/m^2.
[a]Default value applied by the model.

Buffer (Dummy) Links

The Roadsim output does not generate speed, headway, or platoon distribution data for exit links because of the breakdown of the car-following logic when vehicles are leaving the simulated road. To obtain the distribution data for these links (for each direction of travel), a buffer link was added to both ends of the simulated roadway section. Each link was 750 ft (229 m) long, had no horizontal or vertical curvature, and no passing was allowed. This was the shortest possible length that would not affect upstream conditions.

Free-Flow Speed

Free-flow speed is the mean speed at which unimpeded passenger cars (platoon leaders) travel. Roadsim requires a free-flow speed to be specified for the entire roadway or by individual link. An overall free-flow speed was obtained from the field data by averaging the speed of all the platoon leaders.Using this speed in the model's input resulted in mean speeds that were significantly lower than those observed in the field. It was decided to adjust the free-flow speed inputs of individual links to "force" the model mean speeds to be comparable with the observed mean speeds. Therefore, mean speed was a controlled variable. The 30-min data were used to determine this adjustment. The same adjustment was then used in the 60-min data. The average bias per link ranged between 2 and 8 mph (3.2 and 12.9 km/hr). An increase of 5 mph (8 km/hr) in the overall free-flow speed appeared to give similar Roadsim and field results for the mean speed of the overall roadway section.

Traffic Volume

To compare the selected MOEs, a similar number of field vehicle trips and simulation vehicle trips was necessary. Directional hourly volumes for each of the four vehicle types are required input for the model. These volumes are used by Roadsim as an approximation to generate vehicle trips. The actual number of vehicle trips might differ from the input volumes because vehicles that had not traveled the entire roadway when simulation stopped are excluded from the vehicle trip tally and because of the randomness of the vehicle generation logic. To compensate for these, the input volumes were adjusted by trial and error on several Roadsim runs until the number of vehicle trips was similar to the number of trips observed in the field. Therefore, traffic volume was the second controlled variable.

Having the same mean speeds and the same traffic volumes constrains the modeled speed distributions to approximate those observed in the field.

Roadsim Execution

Although the simulation runs would have the same volumes and mean speeds, certain variations were expected because of the randomness of the model's logic. These variations may be observed by changing the "random number seeds" of each run for the initial selection of various parameters, such as headway distributions and driver aggressiveness.

To account for these variations, 10 runs were executed by using different random number seeds. An analysis of variance indicated that 10 mean speeds were not statistically different. Therefore, the results of the 10 runs were aggregated into a single data set for comparison with the field data.

Data Reduction

In most instances, the output generated by Roadsim was in a format that was not directly compatible with the field data. Data manipulation was necessary to convert the simulation data to a comparable format. This inconvenience was a direct result of having to break down the field links into smaller model links and Roadsim's inability to aggregate individual link data into longer links. Enhancing the model to overcome these limiting factors is desirable because restricting the number of horizontal and vertical curves per link results in short links. The user typically is interested in MOEs over long sections of roadway, which might require a large number of links.

Data were manually taken from the Roadsim outputs and manipulated by using the spreadsheet. After all the simulation data had been reduced to the same format as the field data, a statistical comparison was possible.

STATISTICAL COMPARISON

Once the field data and the simulation data had been reduced to similar formats, the MOEs of interest could be compared and analyzed statistically. Because the simulation volume and the mean speed were controlled by varying the input volumes and the free-flow speed entry, an inferential statistical analysis was not appropriate. Instead, the primary MOEs of interest were percent of trucks, percent of vehicles following, cumulative platoon distributions, average platoon size, and the number of completed passes. The collected field data and the simulation data are summarized in Tables 3 and 4.

Traffic Volume

Once traffic volumes had been adjusted to obtain a similar number of vehicle trips, no difference was apparent.

Mean Speed

The mean speed of all vehicles was an adjusted variable. To verify that the model was reasonably adjusted, a t-test at a 95 percent confidence interval was performed. As expected, no statistical difference between the field and Roadsim overall mean speeds was found.

Percent of Trucks

To verify the accuracy of the vehicle generation logic, the percentage of trucks observed in the field was compared with the Roadsim percentage of trucks. No difference was apparent.

Cumulative Platoon Distributions

The cumulative platoon distributions, a good indicator of the level of service of a given roadway, were considered the most important MOE. On two-lane roads, platooning has been proposed as a better method of quantifying level of service than the operating-speed method currently used in the *Highway Capacity Manual (4, 5)*. Platooning characteristics can account for the effect of road geometry and traffic conditions on traffic performance.

The platoon distributions were statistically analyzed by using the Kolmogorov-Smirnov test, which is useful in comparing cumulative distributions that may not be normally distributed. The overall comparison of the field and simulation distributions was found to have no significant difference at a 95

TABLE 3 SUMMARY OF THE 30-MIN DATA

MOE's	Field data		Roadsim data	
	Northbound	Southbound	Northbound	Southbound
Volume (vehicles/hour)	150	152	143[1]	138[1]
Mean speed (mi/h)	54.8	55.4	54.5[1]	55.6[1]
Percent trucks	24	24	22	25
Percent following	44.5	38.5	44.5	38.7
Average platoon size	1.80	1.62	1.83	1.75
Completed passes	2	13	5	12

[1]After adjustment.

percent confidence interval. These cumulative distributions are presented in Figures 3–6.

Percent of Vehicles Following

The percent of vehicles following (vehicles impeded by the vehicle immediately in front) is another MOE that can be derived from the platoon distributions. The results obtained from this MOE for the overall section compared favorably.

Average Platoon Size

This comparison provided another measure of Roadsim's ability to replicate the vehicle grouping that occurred in the field. Comparison of the overall section results indicated a negligible difference between field observations and those obtained through simulation.

Completed Passes

The comparison between the field data and the simulation data of the number of completed passes should be studied carefully. Passing is a traffic measure that reflects the degree of constraint on drivers. Passing opportunities are a function of the opposing traffic and the available sight distance. The lack of passing opportunities translates into an increase in traffic platooning and a decrease in operating speeds and therefore a reduced level of service.

When the number of completed passes is compared, it

TABLE 4 SUMMARY OF THE 60-MIN DATA

MOE's	Field data		Roadsim data	
	Northbound	Southbound	Northbound	Southbound
Volume (vehicles/hour)	138	130	143[1]	136[1]
Mean speed (mi/h)	54.2	54.6	54.7[1]	54.8[1]
Percent trucks	23	25	21	26
Percent following	42.5	41.7	42.0	41.2
Average platoon size	1.74	1.72	1.74	1.71
Completed passes	10	19	10	26

[1]After adjustment.

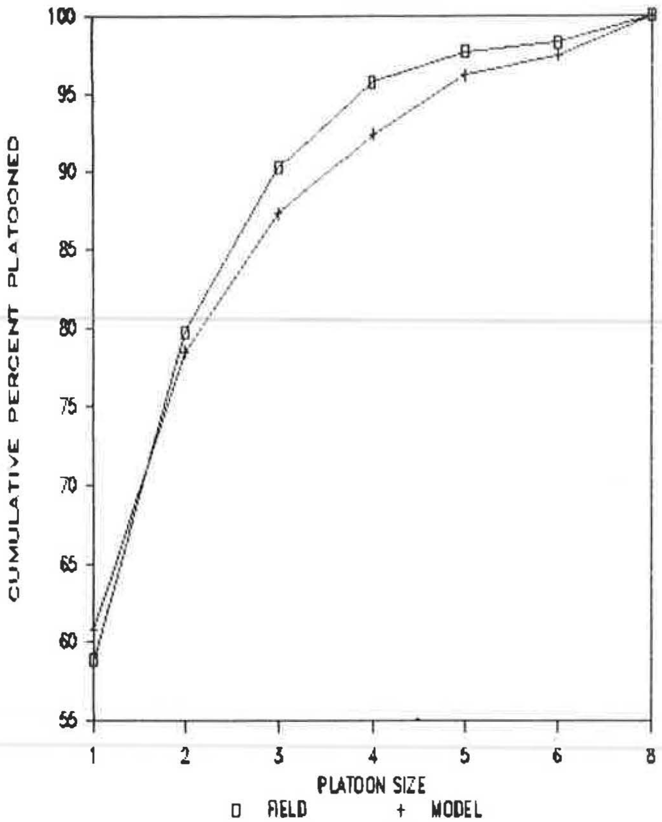

FIGURE 3 Platoon distribution: northbound overall, 30-min data.

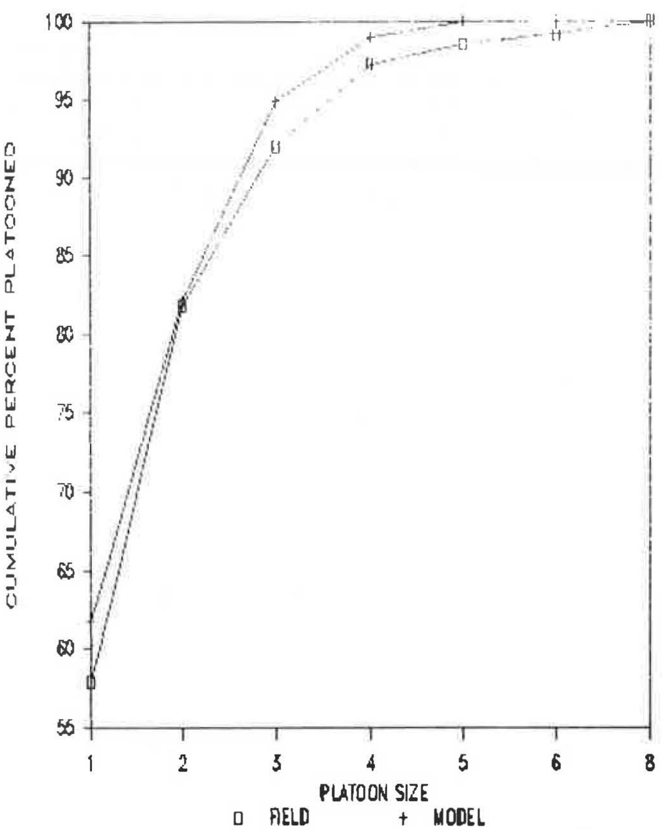

FIGURE 4 Platoon distribution: southbound overall, 30-min data.

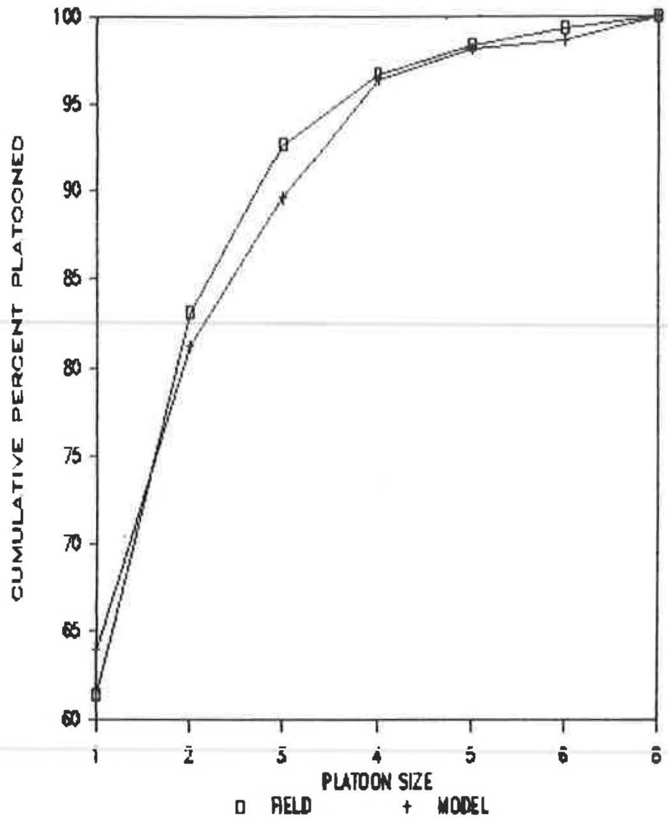

FIGURE 5 Platoon distribution: northbound overall, 60-min data.

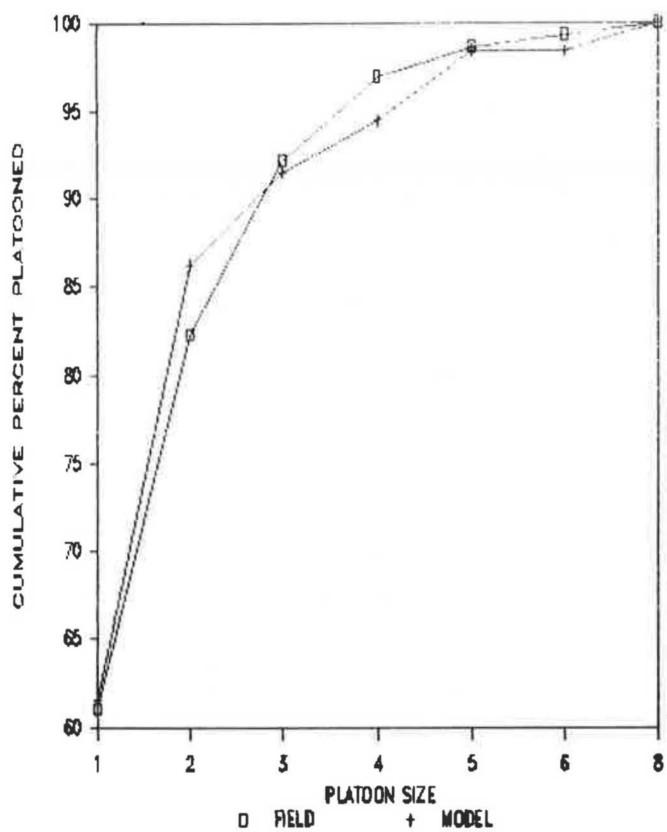

FIGURE 6 Platoon distribution: southbound overall, 60-min data.

should be remembered that there are several factors that influence the decision to pass (for example, driver's aggressiveness and gap acceptance). These factors, although considered in the simulation, cannot be replicated without collection of data for long periods of time. The short data periods being compared in this study were judged insufficient to reach a definite conclusion on the validity of Roadsim's passing logic. Ideally, passes should be compared per unit of time (such as passes per hour), for which longer data periods are desirable. However, the number of completed passes simulated by Roadsim for the available data periods appeared to compare adequately with the field data for the overall roadway section.

SENSITIVITY ANALYSIS

In addition to testing the ability of the Roadsim model to stimulate field conditions, the sensitivity of the model was examined by varying several input parameters to study what effect the parameters have on the mean vehicle speed. The effect on other MOEs was not examined. The parameters studied were the horizontal alignment, the vertical alignment, and a combination of the two. This sensitivity analysis indicated which ranges of the studied parameters significantly affect the mean vehicle speed in Roadsim.

Horizontal and vertical alignments were selected because they are the limiting factors when a roadway section is divided into smaller simulation links. Excluding insignificant geometric features makes possible the use of longer links and simplifies coding the model.

A simple scenario, independent of the field data collection site, was chosen to test these parameters. The following analysis has not been compared with any field data and was undertaken to study the sensitivity within the model.

Horizontal Alignment

Ten simulation runs were executed in which the radius of a curve joining two tangents was varied. The following parameters were held constant during these runs:

- Length of tangents [3,400 ft (1036 m) each];
- Delta of the curve (40 degrees);
- No vertical curvature (0 percent grade);
- Passing allowed on tangents, no passing on curve;
- Free-flow speed [60 mph (97 km/hr)];
- Volume (300 vph, 50–50 directional split); and
- Vehicle mix (20 percent trucks, 0 percent recreational vehicles).

The radius of curvature was varied from 500 ft (152 m) to 3,000 ft (914 m) in increments of 500 ft (152 m). The length of the curve was compared and the superelevation rates were obtained from AASHTO Green Book *(6)*.

Roadsim's results indicated that the effect of curves with a radius greater than 1,500 ft (457 m) was negligible for both automobiles and trucks. This suggests that horizontal curves with radii larger than 1,500 ft (457 m) will not affect the mean vehicle speeds in Roadsim (Figures 7 and 8).

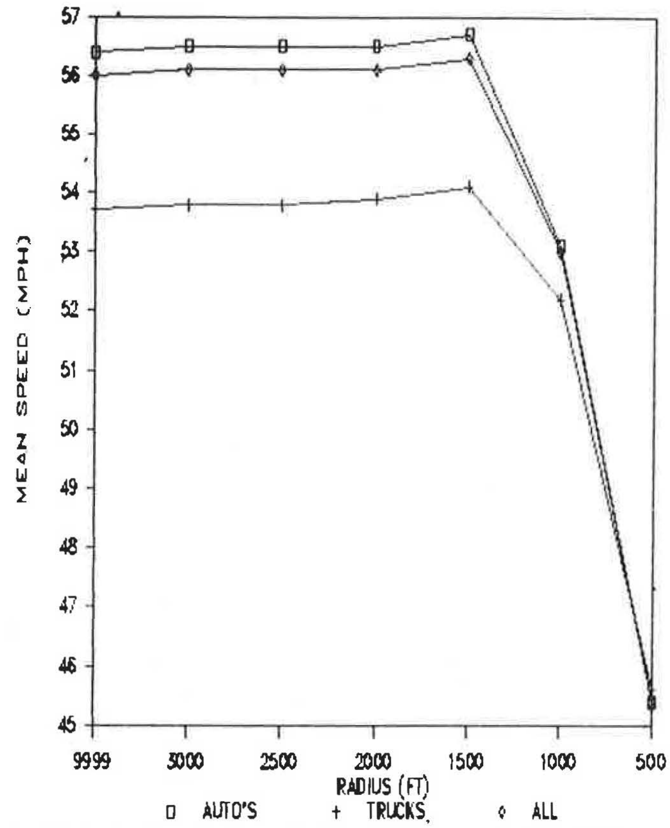

FIGURE 7 Sensitivity analysis: horizontal alignment, right.

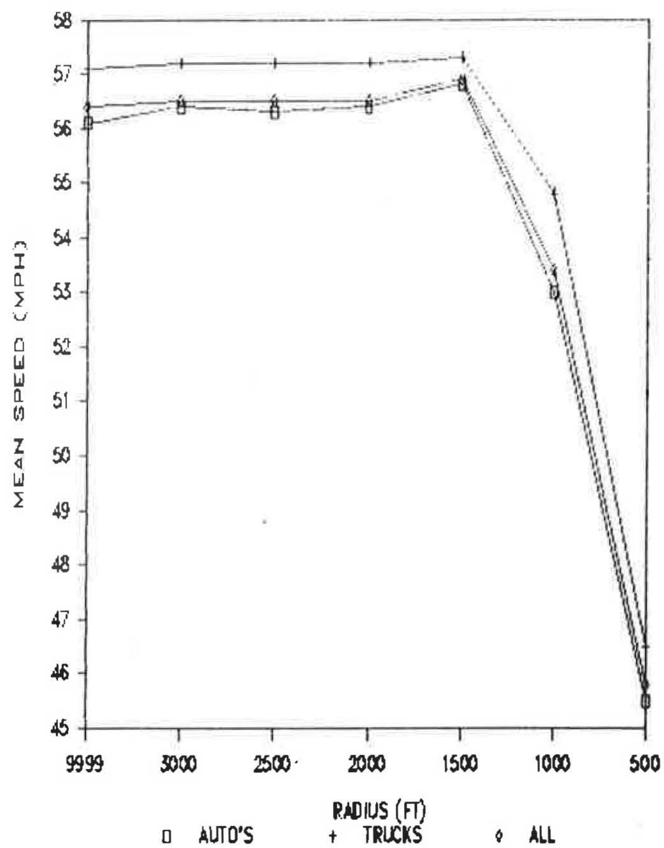

FIGURE 8 Sensitivity analysis: horizontal alignment, left.

Vertical Alignment

Vertical alignment was studied to examine the effect of both the length and magnitude of positive grades. Forty runs were made to study the various combinations. The same parameters just listed remained constant, with the addition of the horizontal curvature (tangent).

The typical truck used had a 266-lb/net-horsepower-ratio (162-kg/kw) power factor and 620-lb/ft^2 (3.03-Mg/m^2) mass-to-frontal area factor.

Results suggested that mean speeds are not significantly affected by grades of 2 percent or less in Roadsim for both automobiles and trucks. At grades of 3 percent and above, the reduction in speed is significant primarily because of the substantial reduction in truck speed on uphill grades. The relatively high percentage (20 percent) of trucks used had a major effect on the overall speeds (Figures 9–11).

Combined Horizontal and Vertical Alignment

Next the combined effect of horizontal and vertical alignment was studied. Having found that grades over 3 percent and curves with a radius of less than 1,500 (457 m) substantially reduced speeds, it was decided not to consider values beyond these thresholds. The worst case of the remaining combinations was selected—a horizontal curve with a 1,500-ft (457-m) radius combined with an uphill grade of 2 percent. Results

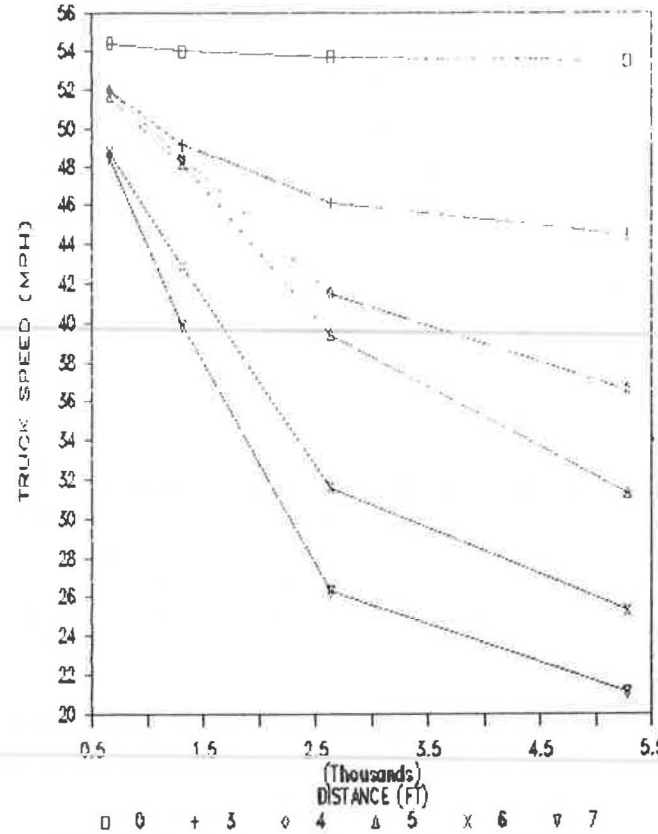

FIGURE 10 Sensitivity analysis: vertical alignment, trucks.

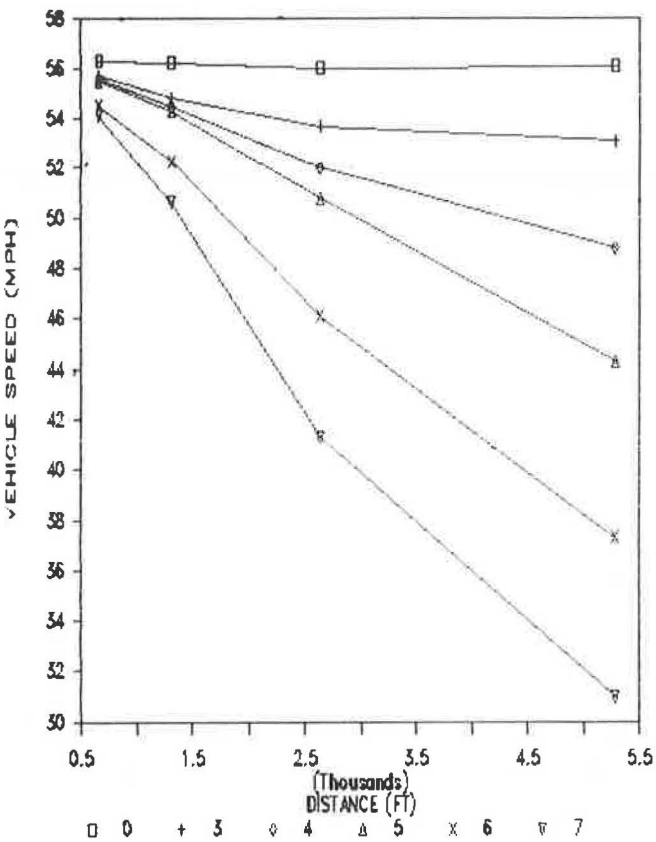

FIGURE 9 Sensitivity analysis: vertical alignment, automobiles.

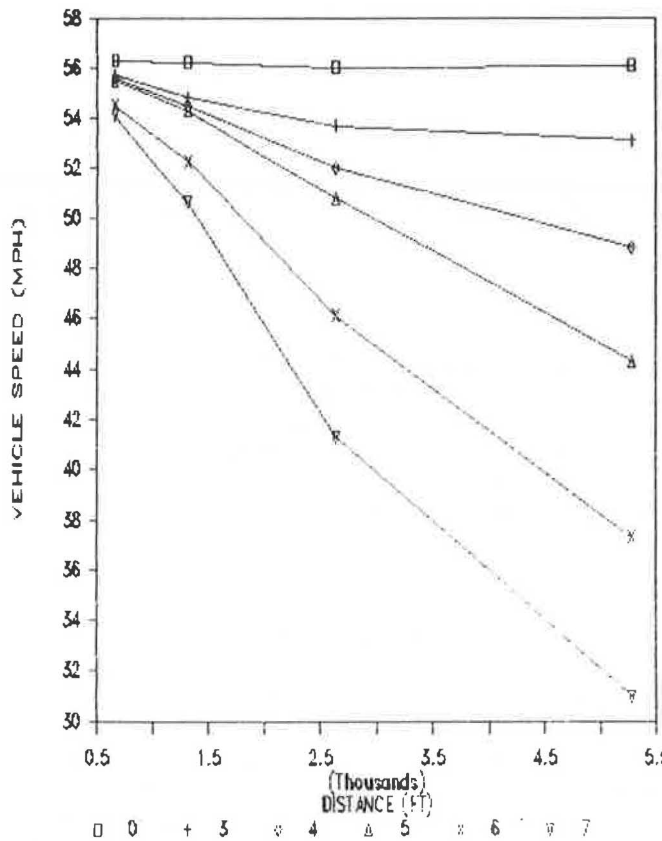

FIGURE 11 Sensitivity analysis: vertical alignment, automobiles and trucks.

showed no apparent difference between the mean speeds on a level, tangent section and the worst-case section.

CONCLUSIONS

On the basis of this comparative evaluation of Roadsim under specific geometric and traffic conditions and the performed sensitivity analysis presented in this paper, the following conclusions may be drawn:

- Roadsim appears to work satisfactorily under the geometric and traffic conditions studied.
- The free-flow speed input appears to be biased. After this input has been adjusted upward, most MOEs compared well with the collected field data for the overall section of road. This bias should be further studied and calibrated.
- Horizontal curves with radii greater than 1,500 ft (457 m) do not appear to significantly affect the overall mean speed of the traffic stream.
- Vertical curves with positive grades of 2 percent or less do not appear to significantly affect the overall mean speed of the traffic stream.
- In its current form, Roadsim can evaluate changes in passing zones, changes in alignment, the effect of volume increases, and the effect of variations in traffic composition.

FUTURE EVALUATIONS AND ENHANCEMENTS

The study described here predicts an optimistic future for Roadsim; however, its full acceptance as a totally valid model is premature. Additional similar studies are necessary to verify the model's performance under a range of traffic and geometric conditions. For example, the performance of Roadsim must be examined in comparison with different real-world traffic bidirectional volumes such as 500, 750, 1,000, and 1,500 vehicles per hour for various terrains (flat, rolling, and mountainous). Further examinations of the free-flow speed input also are needed.

If Roadsim consistently yields results similar to those obtained in the field, the model could be made available for widespread use. However, if it is found that changes and improvements not mentioned in this paper are needed, they could be made when programming upgrades for passing lanes, climbing lanes, and rural intersections are added.

To further assess the functional ability of Roadsim, FHWA would like to receive research reports, results, and recommendations from other users. All comments should be directed to

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McLean, Virginia 22101–2296

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Impacts of the 1984 AASHTO Design Policy on Urban Freeway Design

TIMOTHY R. NEUMAN

The new AASHTO design policy contains many significant revisions and additions that directly address urban freeway design. These additions reflect continuing research on highway safety and operations as well as experience and observation of existing freeways. The latest policy not only updates certain basic design standards but also explicitly recognizes important principles of urban freeway operations and their translation into design guidelines. The focus in this paper is on three important areas in which the new policy will affect urban freeway design: *(a)* general highway design controls and criteria, *(b)* interchange design criteria and standards, and *(c)* freeway systems design principles.

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The focus in this paper is on three important areas in which the new policy will affect urban freeway design: general highway design controls and criteria, interchange design criteria and standards, and freeway systems design principles.

GENERAL DESIGN CONTROLS

Advances in research and evolution of the driver-vehicle system have led to important revisions of many basic design controls. In particular, changes in design for horizontal and vertical alignment, stopping sight distance, and decision sight distance are noteworthy.

Horizontal Alignment

The design curve for the side friction factor has been revised for high-speed facilities. The revision reflects a reassessment of research on vehicle operations on curves. As Figure 1 shows, design values for f are slightly lower for design speeds in excess of 50 mph. The effect is to slightly reduce the maximum allowable curvature for a given design speed and maximum superelevation rate. This represents a marginally more restrictive set of values for design.

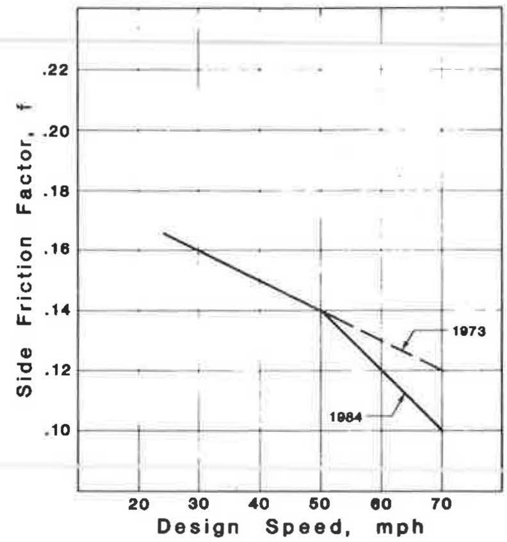

FIGURE 1 Changes in AASHTO design policy for side fraction factors for horizontal curves.

Vertical Alignment

The new policy also revises the basis for critical length of grade, resulting in a slightly more restrictive set of controls. The difference from the 1973 policy is shown in Figure 2. The old design basis was a speed reduction of 15 mph associated with a 400-lb/HP vehicle. The new policy recognizes the more

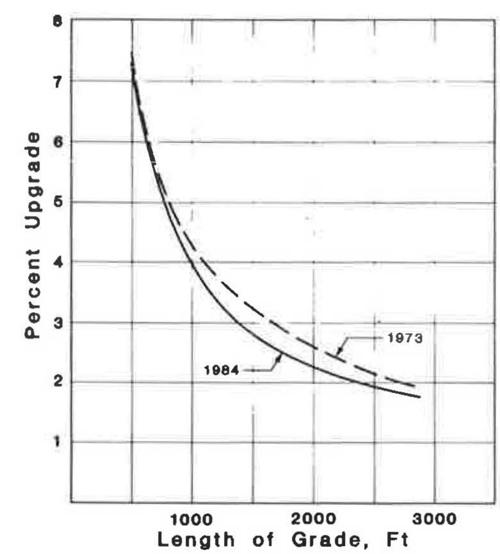

FIGURE 2 Changes in AASHTO design policy for critical length of grade.

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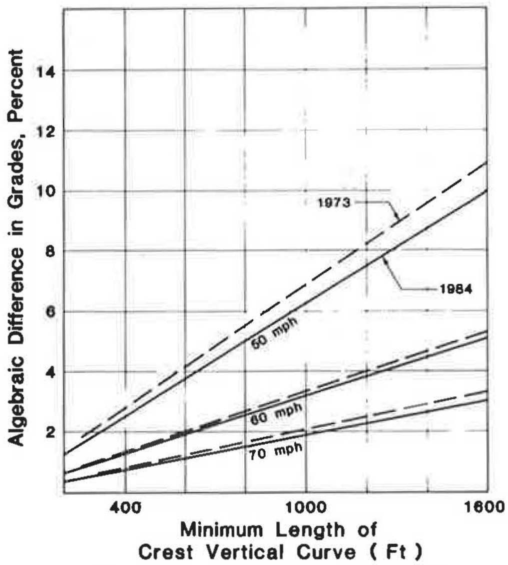


FIGURE 3 Changes in AASHTO policy for crest vertical curve design for desirable stopping sight distance.

powerful vehicle fleet (300 lb/HP) but recommends a more safety-conservative speed reduction of 10 mph. Although the differences are minor, application of the new policy would produce a slightly more restrictive design for grades.

Stopping Sight Distance

Much recent research *(1, 2)* has focused on the need to revise the design for stopping sight distance. Although the new policy retains the basic models for stopping sight distance, many of the design values have been adjusted. These include revisions in the relationship between eye height and object height, slight adjustments to design friction factors for braking, and stopping sight distance requirements and an emphasis on desirable lengths rather than minimum ones. The last assumes operation on wet pavement at design speed rather than at a lower speed assumed under wet conditions.

As noted in the new policy, the previous assumption about driver behavior under wet conditions may not be appropriate (*3*, p.140):

> In prior editions of this book it was assumed that top speeds were somewhat lower on wet pavements than on the same pavements in dry weather. In recognition of this assumption, the average running speed for low-volume conditions rather than design speed was used in formulating the limiting values for minimum stopping distance. This speed is the initial value given in the second column of Table III-1. However, more recent observations show that many operators drive just as fast on wet pavements as they do on dry. To account for this factor, design speed in place of average running speed is used to formulate stopping distance values, as shown by the higher values in the second column of Table III-1.

As with the other design controls, these revisions should produce more safety-conservative designs. The longer crest vertical curve requirements associated with new policy stopping sight distance controls are shown in Figure 3. It may also be noted that horizontal clearance requirements typical of urban freeways (e.g., median barriers, retaining walls, and piers) would also increase.

Decision Sight Distance

The importance of decision sight distance in certain circumstances is emphasized in the new policy. Decision sight distance is recommended at critical locations such as exits, lane drops, and interchanges. The longer distances associated with this design control are shown in Table 1 *(4)*.

INTERCHANGE DESIGN CRITERIA

An integral part of urban freeway design is provision for and design of interchanges. The new design policy highlights important criteria for location and design of interchanges. These criteria reflect operational and safety research and years of observation of existing urban freeways.

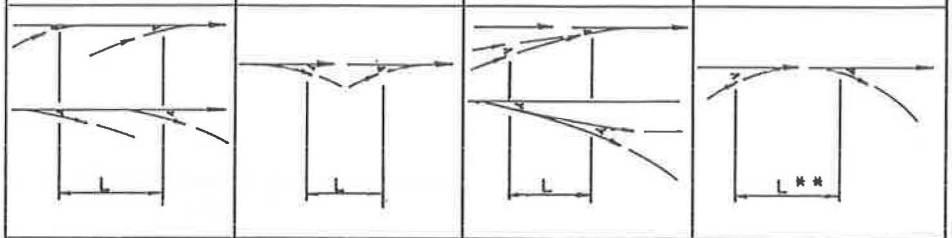
Left-Hand Ramps

The 1973 policy noted the desirability of right-hand exits and entrances. However, their use was not specifically precluded. Safety research *(5, 6)* has unequivocally demonstrated the serious safety problems with left-hand ramps. Consequently, the new policy states that "their use on high-speed, free flow ramp terminals is not recommended" (*3*, p.1031).

TABLE 1 DECISION SIGHT DISTANCE DESIGN CRITERIA *(4)*

	Time(s)			Decision Sight Distance (ft)		
	Premaneuver					
Design Speed (mph)	Detection & Recognition	Decision & Response Initiation	Maneuver (Lane Change)	Summation	Computed	Rounded for Design
30	1.5-3.0	4.2-6.5	4.5	10.2-14.0	449- 616	450- 625
40	1.5-3.0	4.2-6.5	4.5	10.2-14.0	598- 821	600- 825
50	1.5-3.0	4.2-6.5	4.5	10.2-14.0	748-1,027	750-1,025
60	2.0-3.0	4.7-7.0	4.5	11.2-14.5	986-1,276	1,000-1,275
70	2.0-3.0	4.7-7.0	4.0	10.7-14.0	1,098-1,437	1,100-1,450

*RECOMMENDED MINIMUM RAMP TERMINAL SPACING**



MINIMUM VALUES	EN-EN OR EX-EX: FULL FREEWAY	EN-EN OR EX-EX: C-D ROAD OR FWY. DIST.	EX—EN: FULL FREEWAY	EX—EN: C-D ROAD OR FWY. DIST.	TURNING ROADWAYS: SYSTEM INTERCHANGE	TURNING ROADWAYS: SERVICE INTERCHANGE	EN-EX (WEAVING): SYSTEM TO SERVICE INTERCHANGE: FULL FWY	EN-EX (WEAVING): SYSTEM TO SERVICE INTERCHANGE: C-D ROAD OR FWY DIST.	EN-EX (WEAVING): SERVICE TO SERVICE INTERCHANGE: FULL FWY	EN-EX (WEAVING): SERVICE TO SERVICE INTERCHANGE: C-D ROAD OR FWY DIST.
DESIRABLE	1500	1200	750	600	1200	1000	3000	2000	2000	1500
ADEQUATE	1200	1000	600	500	1000	800	2500	1800	1800	1200
ABSOLUTE	1000	800	500	400	800	600	2000	1600	1600	1000

Requirements Noted in AASHTO Policy, 1984

* BASED UPON OPERATIONAL EXPERIENCE AND NEED FOR FLEXIBILITY

**ALSO TO BE CHECKED IN ACCORDANCE WITH PROCEDURE OUTLINED IN THE HIGHWAY CAPACITY MANUAL, 1965 (LARGER OF THE VALUES TO BE USED)

FIGURE 4 Interchange ramp spacing controls.

Ramp Spacing Controls

High-volume freeway mainline and ramp traffic demands create special problems in interchange design. The new policy presents design guidelines for ramp spacing based on those shown in Figure 4. These guidelines reflect the importance of ramp location in distribution of volumes and optimization of traffic flow. Investigation of many existing urban freeways has shown that capacity and operational problems are often due to violation of these ramp-spacing criteria.

Interchange Selection

Years of experience have contributed to revised guidelines on freeway interchange selection. An effective set of guidelines for considering alternative interchanges is shown in Figure 5. For service interchanges, simple diamonds or partial cloverleafs are usually optimal. Such interchange types have been shown to optimize both freeway ramp movements and arterial-intersection operations. System interchange types vary, but right-hand exits are always incorporated with one or more

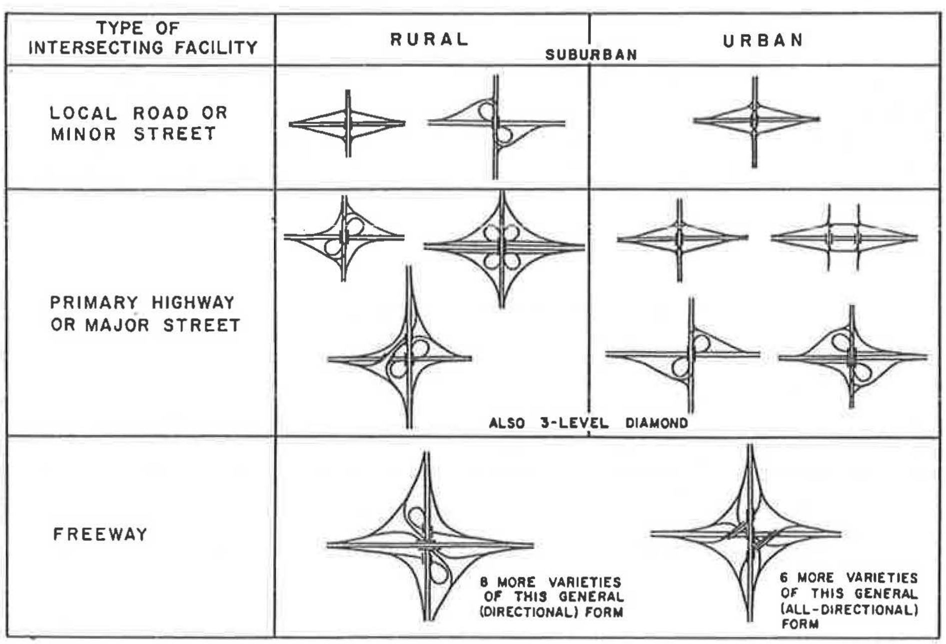


FIGURE 5 Guidelines for selection of interchange types on freeway facilities.

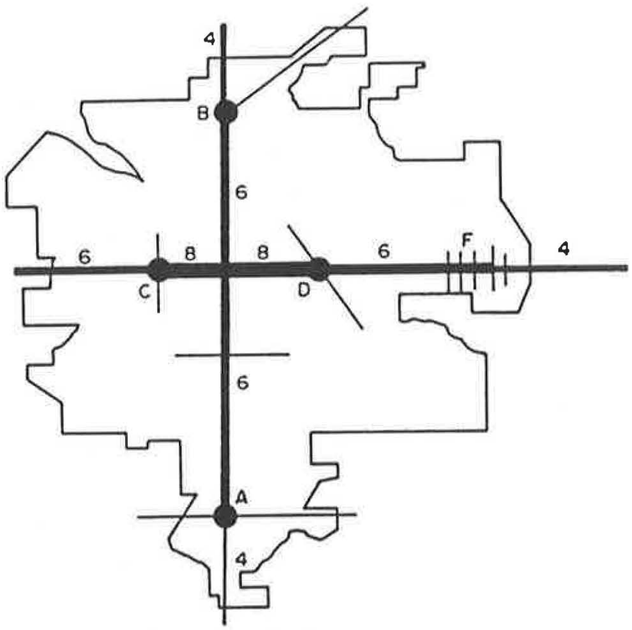


FIGURE 6 Schematic of basic number of freeway lanes.

direct connections. Minimizing or eliminating weaving sections within system interchanges is also a primary consideration.

In particular, it is pointed out in the new policy that cloverleaf interchanges in urban areas are inherently inadequate. Problems associated with weaving between the loop ramps necessitate the use of collector-distributor roads and greater distances between the loops. Such requirements result in extensive right-of-way needs, which are usually impractical or not cost-effective. As a result, the full cloverleaf is identified as being inappropriate for most urban freeway applications.

SYSTEMS DESIGN POLICIES

Perhaps the most important areas of the new policy deal with treatment of urban freeways as systems. The material presented here is not new to many freeway design and operational practitioners. Its inclusion in the new policy, however, is a significant recognition of the importance of these principles.

Basic Number of Continuous Lanes

Figure 6 shows the principle of basic number of lanes. Good operation of an urban freeway system requires that each facility be assembled logically with respect to basic number of lanes. This generally means an increase in the basic lanes as the facility approaches the highest-density, central areas of a city. Basic lanes should be continuous, enabling through drivers to remain on the freeway for long distances without having to change lanes. A constant number of basic freeway lanes should be provided for a meaningful distance regardless of minor variations in forecast traffic flow. Serious, costly operational bottlenecks have occurred on many existing freeways because planners sized the freeway strictly according to expected design-year traffic and ignored the principle of basic lanes.

Lane Balance and Continuity

Many operational problems on existing urban freeways are directly attributable to a lack of lane balance at exits and failure to maintain lane continuity. Leisch *(7)* has demonstrated the operational benefits of these principles (Figure 7). In brief, these include meeting driver expectations, accommodation of periodic short-term volume fluctuations, and minimizing lane changing.

Interchange and Ramp Uniformity

Maintaining interchange uniformity is consistent with designing for driver expectations. Single exits on the right at all interchanges satisfy such expectations. Consistent use of similar or identical interchange forms or ramp arrangements also addresses this principle.

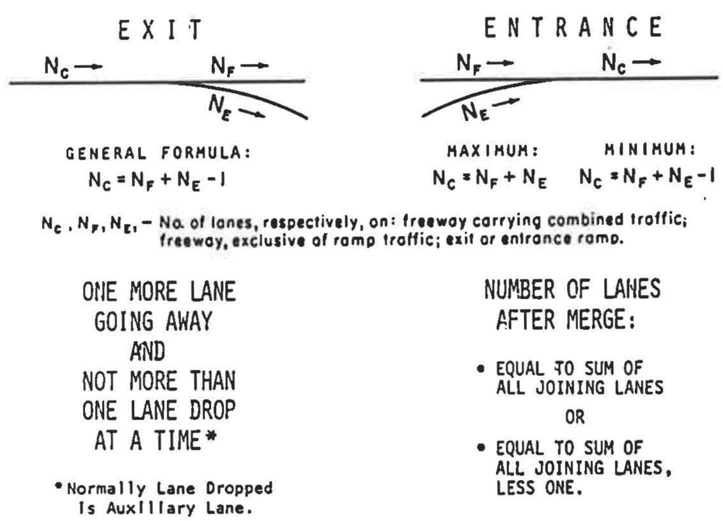


FIGURE 7 Principle of lane balance.

Importance of System Design Principles

Many existing urban freeways operate at or near level-of-service E for long periods of the day. Reconstruction solutions to improve this level of service and also accommodate expected traffic growth are extremely costly. In many cases, the practical limits of reconstruction will produce level-of-service D or E in the design year. For such cases, application of the systems design principles becomes essential. Every effort should be made to ensure smooth, orderly exiting and entering, and to limit lane changing to only that required for navigation. Designing for driver expectations and achieving consistency in the freeway's operations will produce marginally higher capacity. When the freeway is operating near possible capacity and breakdowns reflect upstream for several miles, such marginal improvements produce significant total benefits to the driving public.

IMPLICATIONS OF PRINCIPLES AND CRITERIA IN NEW POLICY

Reconstruction of congested and outmoded urban freeways has emerged as the greatest challenge currently facing the highway design profession. Older urban freeways, designed and built with imperfect knowledge of high-volume operations, do not function adequately. Moreover, their problems often stem from interchange and ramp design, and not merely from an inadequate number of lanes. Planners and designers must recognize that appropriate reconstruction solutions require more than new pavement, added lanes, and selected safety improvements. Almost without exception, existing urban freeways fall short in a comparison with the design principles and criteria discussed in the new policy.

A systematic approach to freeway reconstruction is clearly indicated. The following brief outline illustrates how the new AASHTO policy should be applied in evaluation and reconstruction of an existing urban freeway.

Consider Existing Geometry

Existing horizontal and vertical alignment may no longer meet an acceptable design standard. This does not necessarily mandate expensive geometric changes. However, it is clearly appropriate to assess the nature and extent of each geometric deficiency. Evaluation of accident patterns, field inspection, and engineering analyses should be performed to determine any need to upgrade outmoded alignment. This is not only good engineering, but it is also a necessary step toward protecting the responsible agency from future tort liability claims.

Perform Complete Capacity Analyses

Urban freeway operations are not limited to uninterrupted flow conditions. Ramp locations and sequencing may, in fact, be the controlling factor in a bottleneck situation. Critical analysis of ramp location controls, along with ramp and weaving level-of-service analyses, may reveal solutions to exisiting operational problems.

Develop System Solutions

For most freeway corridors, all elements interact to influence operations. Under heavy traffic, even minor localized flaws may cause corridorwide breakdowns. Under such conditions, the principles of lane balance and lane continuity become essential. Appropriate interchanges and ramp spacing are as important as good cross section or alignment design in maximizing capacity as well as safety.

WHAT HAS NOT CHANGED

The presentation thus far has focused on changes in the policy and their significant implications for urban freeway design. Although much has changed, it is important to note certain areas that, for very good reasons, remain unchanged. These include the concept of design speed and its application to freeways and cross-sectional design criteria for freeways.

Design Speed

The design profession and driving public are by now adapted to the politically established national speed limit of 55 mph. Despite the apparent permanence of the 55-mph limit, the appropriateness of design speeds of 60 and 70 mph for freeways remains in the policy (*3*, p.63):

> Although a lower design speed may satisfy the majority of this current slower traffic [i.e., that induced by the 55 mph limit], a design speed of 70 mph should be maintained on freeways, expressways and other major highways.

Cross Section

Much recent experimentation has taken place with cross-section revisions to increase freeway capacity. Shoulder conversions to additional conventional or high-occupancy-vehicle lanes, lane-width narrowing to 10 or 11 ft, and combinations of the two have been tested in many places *(8)*. Despite the apparent success of such innovative designs, the new policy maintains a constant stance on cross-sectional dimensions. It is clearly stated (*3*, p.631) that "through-traffic lanes should be 12 feet wide" and that "on freeways of six or more lanes, the usable paved width of the median shoulder should be 10 feet and preferably 12 feet where the truck traffic exceeds 250 DHV."

Adherence to such strict dimensions is not intended to discourage innovations in cross-section treatment. It does, however, indicate the need for serious study and evaluation of trade-offs before implementation of restricted-width designs. Designers should not easily arrive at decisions to compromise the comfort, convenience, and safety provided by full-width designs.

SUMMARY AND CONCLUSIONS

The 1984 AASHTO policy presents a challenge to planners and designers concerned with urban freeways. Revisions to many basic design controls (horizontal alignment, vertical alignment, stopping sight distance) mean that many existing freeways no longer meet current standards. Careful consideration of substandard geometry must accompany major rehabilitation or reconstruction of such freeways.

In addition, the policy clearly charts the course for a systematic approach to freeway and interchange design. Again, many older freeways require substantial planning and redesign to accommodate the operational objectives of the principles discussed in the new policy.

Finally, the policy maintains a proper stance toward the basic characteristics of freeways. The continued use of 60- and 70-mph design speeds and full-width cross-sectional elements is recommended. This should ensure the continuation of freeways as the safest, most efficient elements of the highway system.

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Impact of the AASHTO Green Book on Highway Tort Liability

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The new AASHTO design policy for highways and streets (Green Book) includes new and revised concepts on geometric design that reflect changes in design philosophy, design vehicles, roadside safety features, and driver behavior. Those concepts and how they affect highway tort liability are addressed. The consequences of design flexibility and functional roadway classification are presented; the implications of design consistency and driver expectancy are also discussed.

Many city, county, and state governments in the United States have been forced to devote extensive time and energy to defending themselves against highway tort litigation. (A tort is defined as a civil wrong, as opposed to criminal activity, and is normally classified as negligence.) Highway tort actions normally are based on plantiff accusations that the governmental agency (or its employees) responsible for design, maintenance, and operation of a roadway was negligent in performing its duties, and that this negligence caused the plaintiff to have a traffic accident that resulted in serious injury (or death). The plaintiff sues the agency in hopes of collecting an award (money) for his damages (injuries).

Proof of negligence must be clearly demonstrated by the plaintiff. One of the most effective methods to establish this proof is to show how the agency failed to design, maintain, or operate the roadway according to recognized standards, operational procedures, or policies.

Although clearly identified as design criteria policies or guidelines, the AASHTO publications entitled *A Policy on Geometric Design of Rural Highways* (Blue Book) (*1*) and *A*

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Policy on Design of Urban Highways and Arterial Streets (Red Book) *(2)* have been consistently accepted by the courts as the nationally recognized standards for highway and street design. The 1984 AASHTO publication, *A Policy on Geometric Design of Highways and Streets* (Green Book) *(3)*, essentially replaces the Blue and Red books. Hence, it is sure to be touted as the latest edition of the nationally recognized design standards for highway and street design.

Because the Green Book contains some new and revised design concepts, there may be new areas of exposure to highway tort liability. An attempt to identify those new areas and explain methods for reducing the risk of lawsuit involvement in those areas is made in this paper.

STATE-OF-THE-ART DESIGN CRITERIA

The Green Book is a collection of design criteria pertinent at the time of its writing (1984); hence, the publication is considered representative of the state-of-the-art design criteria. It must be emphasized that the Green Book is not a publication of design standards. In the foreword of the book, it is clearly stated that "the intent of this *policy* is to provide *guidance* to the designer by referencing a recommended *range of values* for critical dimensions" *(3)* (italics added by the authors of this paper for emphasis). The Green Book, therefore, does not present a series of precise roadway design standards. Instead, it may be defined as a policy of design guidelines that recommends various ranges of dimensional values for consideration in design.

A common argument by the plaintiff in a tort case is that the roadway in question did not meet current design standards. This statement is often true. Most older roadways do not have the wide travel lanes, wide stabilized shoulders, or the bridge widths currently identified in the ranges of design values in the Green Book. In the foreword of the Green Book, this argument is clearly addressed *(3)*:

> The fact that new design values are presented does not imply that existing streets and highways are unsafe . . . . This publication is intended to provide guidance in the design of new and major reconstruction projects. It is not intended as a policy for resurfacing, restoration, or rehabilitation (R.R.R.) Projects.

As design standards change, there is no requirement to reconstruct all roadways to meet the new standards. Such a requirement would be impossible to satisfy. Roads would never be completed because all roadways would have to be constantly upgraded. Furthermore, the funding requirements would be unbelievable.

All roadways cannot be continually upgraded to satisfy changing design criteria. The following is a good illustration of this principle. Suppose that a city government decides all residences must henceforth have a 30-ft setback from the property line instead of the 25-ft setback established by city ordinance. Does it make sense to require all residences having a 25-ft setback to be relocated an additional 5 ft away from the front property line?

The same principle could be applied in the automotive industry. Even though new safety design features are constantly being developed, automobile manufacturers do not recall all their vehicles back to the plants for reconstruction every time a new design standard is introduced. Of course, on occasion, the manufacturer has to recall some automobiles to correct some deficiency. Similarly, to protect drivers who lose control of their vehicles, many state highway departments have upgraded roads by installing breakaway signs to replace fixed roadside signs and crash cushions (attenuation devices) in freeway gore areas.

The plaintiff's argument that the roadway in question did not meet current standards is best countered with the statements in the foreword of the Green Book. In support of the governmental agency's defense position, the Green Book may be used to illustrate that the roadway involved in the litigation actually met current design criteria and guidelines specified in the 1984 publication. The fact that a roadway designed in the 1950s still satisfies the state-of-the-art design criteria in the manual (Green Book) is strong supportive evidence that the agency is building and maintaining modern roadways.

DESIGN FLEXIBILITY

The Green Book attempts to avoid specifying exact geometric design dimensions for highways and streets. Instead, design guidelines and ranges of values are provided to allow some flexibility in the design of highways and streets. The roadway designer does not have to resort to prescribed designs and is allowed freedom for innovation. However, freedom may be viewed as a two-edged sword.

Most design engineers enjoy freedom of discretion when preparing a design for a new roadway. Available design criteria (including those in the Green Book) are most helpful in providing general (and sometimes specific) guidelines for dimensional design. It is helpful to recognize that when restrictions are placed on design options (e.g., narrow rights-of-way), minimal dimensions are still considered satisfactory and safe according to the Green Book guidelines.

However, discretionary freedom may also work against the original designer. Various opinions may be developed by other design engineers, and each of these opinions may be viewed as satisfactory according to the range of the design guidelines. Specific design criteria are easy to defend in a courtroom: the roadway design was either right or wrong. Ranges of design values present a more difficult defense position. Many design decisions could be viewed as satisfactory according to design standards, but the interpretation of choice falls into a gray area.

Some engineers believe that discretionary decisions are immune from tort liability. This belief is incorrect. Anyone (or any public agency) may be sued by anybody for anything at any time. (Of course, winning the suit is not always easy, but filing a claim is extremely easy and inexpensive.)

Generally, discretionary acts are design oriented and enjoy the protection of immunity from tort liability. However, there are exceptions. The courts may find that design immunity is not valid in cases where the design was not prepared with appropriate care, the plan was so obviously dangerous that a person acting prudently would not have approved it, or the design was dangerous or unsafe after its implementation (e.g., the design was simply not done correctly) and the responsible agency had received notice of that fact *(4)*.

In presenting his claim, the plaintiff may hire an expert witness to testify that the roadway should have been designed differently and that if it had been designed differently, the accident in question would not have happened. Of course, the expert has the benefit of hindsight. However, the jury will be presented with the alternative design by the expert and will have to compare it with the actual design selected by the public agency. If both designs satisfy the design guidelines and criteria of the Green Book, the jury may have difficulty understanding why the alternative design was not selected.

In selecting a proper roadway design it is important to ensure that the design features satisfy the guidelines contained in the Green Book and that the design selected is satisfactory for the conditions and restrictions presented. Moreover, the basis for the design selection should be well documented to provide justification for the selected design and evidence for legal defense, if necessary. This documented evidence is the strongest argument that can be presented to counter the claims of the plaintiff's expert witness. Although not able to benefit from hindsight, the decision-making agency should document the reasoning for the design selection on the basis of projected traffic conditions. This decision-making process could be explained in the courtroom by a witness for the defense. However, the testimony is much stronger if a document is presented that describes the decision-making process and gives a preparation date several months (or years) before the relevant accident.

NEW AND REVISED CONCEPTS

General

The basis for the design modifications and new concepts in the Green Book may be found in the changes in philosophy or attitudes of the highway engineering profession, advancements in technology, and proven experiments and successful operations of new roadway geometric concepts.

Since the publication of the Blue and Red books, changes in philosophy and attitudes of highway design engineers have altered the concepts of design criteria. Probably the most significant design modification in the Green Book is based on a change in philosophy. Average daily traffic (ADT) volumes were used as the fundamental basis for design in the Blue and Red books. Roadways were designed for the design ADT volume. In the Green Book it is argued that roadways should be classified according to their function or use and then be designed to fulfill that function. Hence, the functional classification of roadways has become the initial requirement for design.

Additional design vehicles have been added to the Green Book, and new design topics have also been added. These additions reflect a desire to be more thorough with design standards.

A concentrated effort was made in the Green Book to develop design criteria that emphasize consistency. Significant attention has been devoted recently to the human factors element of highway design. Humans are creatures of habit and perform more efficiently when they work (or operate) in a familiar environment. In performing the driving task, humans have a tendency to expect certain operational conditions and roadside features. Hence the need to acknowledge driver expectancy is encouraged in highway designs. Design consistency to satisfy driver expectancy is addressed throughout the text of the Green Book.

The new guidelines are very safety oriented and deal extensively with the "forgiving roadside" concept. At the same time, the Green Book includes geometric design and operational concepts that were developed during the past 10 to 20 years. An example is the two-way left-turn lane, which was not emphasized in either the Blue or Red book.

Technological changes have also had an impact on the Green Book. Since the oil embargo of 1973, American cars have been getting smaller, and trucks have been getting larger. Cars are no longer so powerful as they were 12 years ago, but trucks have become more efficient and more powerful. Trucks can now compete very well in acceleration and speed with the passenger car on high-speed, relatively flat roadways. Design modifications in the Green Book reflect these developments.

A major roadway design change related to the smaller car involves the lowering of driver eye height. For design purposes, the Blue and Red books used a driver eye height of 3.75 ft. The Green Book uses a driver eye height of 3.50 ft. Although this change may appear to be insignificant, it does affect sight distance calculations.

This overview of the new and revised concepts of the Green Book has identified four major areas of concern relative to potential tort litigation:

1. Functional classification,
2. Design vehicles,
3. Driver expectancy, and
4. Safety design.

Functional Classification

Functional classification groups rural highways and urban streets into categories according to their character of service. Initially, roadways are classified as either rural or urban, depending on their location. According to the Green Book, urban areas are "places within boundaries set by responsible State and local officials having a population of 5,000 or more" *(3)*. Urban areas are further categorized as either "urbanized areas" (having a population greater than 50,000) or "small urban areas" (having a population between 5,000 and 50,000). Rural areas are those locations that do not qualify as urban areas.

Roadways are classified within urban and rural areas as follows:

Urban	*Rural*
Principal arterial	Principal arterial
Secondary arterial	Minor arterial
Collector street	Major collector
Local street	Minor collector
	Local road

Roadways fall into one of three general categories: arterials, collectors, or local streets and roads. The hierarchy of classification indicates that arterials are the major highways and thoroughfares, and the local streets and roads are the least important roadways.

Some roadways can be easily classified into one of the categories. Obviously, Interstate highways and freeways are primary arterials. Other roadways may satisfy the descriptions of one of the other classifications without difficulty. However, some roadways are difficult to classify, and engineer discretion is required.

Population growth may cause some rural roadways to become urban roadways, but the classification guidelines are based on population figures and are very specific. Courts would have no problem discerning which classification applies. However, determining the appropriate category for those roadways that are difficult to classify causes some concern. For the higher functional classifications, design guidelines suggest wider roadways, more rights-of-way, stabilized shoulders, shorter curve radii, and generally a better or more sophisticated roadway. In litigation, a plaintiff will argue that the roadway in question should have been given a higher classification, and his expert witness will provide evidence as to why the higher classification was warranted and why different design criteria should have been used. The public agency will defend the design criteria used in constructing the roadway. Once again, the court is presented with arguments that are matters of opinion, and it must determine which argument is most valid.

The best defense for the public agency is to provide documentation addressing the basis for its classification system and the reasoning for selecting the functional classification category and the pertinent design criteria. Because the functional classification approach may be new to some design engineers, additional development of evaluation factors will be forthcoming. Until a more precise process is developed, design engineers must select the most reasonable classification categories for their existing and future roadways and be careful to document their decisions for purposes of justification and legal defense.

Design Vehicles

Design vehicles listed in the Blue and Red books included the following:

1. Passenger car,
2. Single-unit truck,
3. Single-unit bus,
4. Semitrailer intermediate,
5. Semitrailer combination, and
6. Semitrailer–full trailer combination.

Four additional design vehicles have been included in the Green Book:

7. Articulated bus,
8. Motor home,
9. Passenger car with travel trailer, and
10. Passenger car with boat and trailer.

The last three reflect AASHTO's recognition of the need to design unique roadways for unique vehicles.

The 1982 Surface Transportation Assistance Act allowed for larger vehicles than those just listed. They were not included in the Green Book because AASHTO decided that the time required to reflect the changes to the Green Book necessitated by the larger vehicles would excessively delay its publication date. A supplement to the Green Book that will address the additional design requirements for the larger vehicles is currently being developed.

What is important from a tort liability viewpoint is the need to recognize that the larger vehicles and the vehicle combinations (e.g., passenger car with trailer) require larger turning radii, and design engineers should consider these vehicles when selecting the geometric design criteria for roadways. This does not mean that all roadways must be designed to accommodate the larger vehicles. Such overdesigning is unnecessary and economically wasteful. But it does suggest that attention should be given to designing intersections, ramps, driveway entrances, and all roadway grades to minimize adverse operational effects on the larger vehicles. This need is especially important for roadways serving a significant traffic demand.

Some roadways require special design considerations for larger vehicles, recreational vehicles, or both. Roadways and intersections within or near industrial parks should be designed to accommodate the larger trucks that are expected to travel to and from the park. Roadways and intersections built to provide transport to recreational sites (campgrounds, lakes, state and national parks, etc.) should accommodate motor homes and the passenger car and trailer combinations.

Suppose that an accident involving a motor home on a roadway serving a popular state campground and park facility results in a tort claim. The plaintiff may contend that the geometric design of the roadway did not accommodate the required wide turn of a motor home and that this design deficiency caused the accident. It would be hard to defend the suit if no special consideration had been given to designing for that particular vehicle and if the state (or responsible governmental agency) knew that the roadway was built primarily to serve recreational vehicles.

Driver Expectancy

Expectancy "relates to the process in which an individual with an established set of ideas and concepts is presented with a stimulus of some type . . . and responds in some fashion to this stimulus" *(5)*. Driver expectancy causes a driver to respond in a set manner to a traffic-related situation on the basis of previous experience. For example, because of consistent use of standardized signs, motorists expect to see STOP signs that are red and octagonal shaped, not circular and blue or rectangular and black.

The Blue and Red books did not emphasize design based on driver expectancy; however, the Green Book devotes significant attention to it. Some examples of driver-expectancy design criteria are the following:

1. Lane balance: Generally, lane balance is achieved in merging areas by maintaining the same number of through lanes approaching and leaving the merging areas and by gradually transitioning lane drops downstream of the merging areas. Such design is strongly encouraged in the Green Book.

2. Major route emphasis: When two roadways reach a point of bifurcation, the most direct connection should be the continuation of the most important route. Recommended freeway design criteria in the Green Book suggests this treatment.

3. Consistent freeway ramp design: Ramp design consistency (e.g., the same type of ramp along a section of freeway) and providing all freeway exits on the right are major driver expectancy design criteria included in the Green Book.

4. Design by functional classification: One of the purposes of establishing roadway design by functional classification is to develop consistency in roadway design features so that drivers will learn to recognize the function of a facility according to its geometrical configuration. Hence, the consistency in design will help to develop driver expectancies.

Another design feature addressed in the Green Book is the concept of decision sight distance, or "the distance required for a driver to detect an unexpected or otherwise difficult-to-perceive information source or hazard in a roadway environment that may be visually cluttered, recognize the hazard or its threat potential, select an appropriate speed and path, and initiate and complete the required safety maneuver" *(3)*.

Decision sight distance is required in locations where driver expectancy is violated or where there is potential driver error in receiving information, making a decision, or controlling the vehicle. These locations include interchanges, intersections, lane drop locations, and areas having significant "visual noise." The values for decision sight distance contained in the Green Book reflect driver decision and reaction times in the range of 10 to 14 sec.

The concept of driver expectancy creates additional and unique exposure to tort liability. Because the Green Book emphasizes driver expectancy design, any roadway designed according to the guidelines should accommodate driver expectancy considerations. Plaintiff claims that indicate driver confusion or violations of driver expectancy may be dealt with in one of two ways.

First, an argument may be presented by the defense that driver expectancy was not violated, and factual proof supporting this contention may be provided. Second, the defense may recognize that the roadway condition may be contrary to driver expectancy (e.g., a left-side freeway exit), but that sufficient information was provided to establish adequate decision sight distance. In either situation, the driver expectancy concept may be difficult to defend because the human factors element plays such an important role. Furthermore, the human element is more clearly understood by a jury than engineering principles and technical computations.

Safety Design

The Green Book is very safety oriented. New design concepts and modified design criteria are all based on safety-related research, technological changes, or traffic operational findings. Design criteria and guidelines contained in the Green Book reflect wider, straighter, and flatter roadways, more recovery area for out-of-control vehicles, and greater built-in factors of safety than those in either the Blue or Red book.

Some specific safety design features included in the Green Book are the following:

1. Increased stopping sight distances: Driver eye height for design purposes was lowered from 3.75 to 3.50 ft to reflect the trend toward smaller American cars. Also, the design friction factors (or drag factors) used to determine braking distances were lowered to increase the design factor of safety. Because of these modifications, design stopping sight distances were increased for all types of roadways.

2. Increased vertical curve lengths: The reduction in driver eye height and the increase in stopping sight distances have resulted in longer vertical curves or, in essence, "flatter" curves.

3. More gentle horizontal curves: The increase in design stopping sight distance has necessitated longer radii for horizontal curves. In other words, minimum curve radii have been increased for various speeds.

4. Design criteria for emergency escape ramps: Among the new design concepts introduced in the Green Book are design criteria for emergency escape ramps. These ramps are designed primarily to stop out-of-control trucks on roadways in mountainous terrain.

All design criteria and guidelines in the Green Book reflect the most recent roadway safety design innovations available to the roadway designer. Consequently, roadways designed and constructed in accordance with the Green Book will be considered safe as well as efficient. Because tort claims result from traffic accidents, it stands to reason that fewer accident occurrences because of safer roads will result in fewer tort claims. Safe roadways do not eliminate accidents, but they help to minimize the number of occurrences and to reduce severity levels.

Governmental agencies can minimize their risk to tort litigation by ensuring that their new and reconstructed roadways are designed in accordance with the Green Book. In fact, one of the strongest defense positions that can be taken in a tort lawsuit is to demonstrate proof of conformance with recognized design standards, criteria, and guidelines that were in effect at the time of the design and construction of the roadway in question.

If severe design or operational restrictions prohibit a governmental agency from designing or constructing a new or reconstructed roadway in accordance with the Green Book, the agency should ensure that the decision-making process followed in deciding not to comply with the Green Book is documented and that the reasons why compliance was not possible are explained. Such documentation, usually in the form of standard policies and procedures, is necessary for possible legal defense. Without such evidence, the governmental agency may find itself in an extremely vulnerable position.

SUMMARY

New design concepts and modified design criteria in the Green Book are all based on safety-related research or operational findings. Applications of the design criteria and guidelines contained in the Green Book will provide safer, more efficient, and more comfortable roadways. Hence, the safer roadways will help to reduce traffic accidents, which in turn will help to minimize tort-related lawsuits resulting from accidents. Compliance with the Green Book is an effective method of reducing highway tort liability.

At the same time, there is a tendency to avoid establishing precise design criteria in the Green Book. Instead, ranges of design values are provided, which affords the design engineer greater flexibility in selecting the design features of a roadway. This design flexibility may be viewed as a two-edged sword. It allows the design engineer to be innovative and provides freedom to exercise discretion. However, the plaintiff in a tort lawsuit can present alternative designs that are claimed to prevent the tort-related accident. Both designs could satisfy the design criteria and guidelines of the Green Book. The jury faces a dilemma in trying to determine whether the original design was inadequate and therefore hazardous.

The best defense for a public agency is to document the decision-making process when selecting the design for new or reconstructed roadways. If the design does not comply with the Green Book, it is imperative that the reasons for noncompliance be explained and documented for use as potential evidence. Because a multitude of potential designs can be developed in accordance with the Green Book, it is important that the discretionary decisions made by the design engineer also be documented. This documentation could provide the primary evidence necessary to successfully defend a future tort lawsuit.

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New Approach to Geometric Design of Highways

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A basic deficiency in the current practice of geometric highway design is a lack of sensitivity to traffic volume, traffic composition, and construction and user cost factors. Current practice is based on a deterministic approach, whereas the factors involved in the geometric design process (e.g., speed, friction, reaction time) are stochastic in nature and vary among road users. The current approach employs only a single value to represent each factor. Criteria that are used to generate these representative values are not made explicit. An alternative approach to geometric design of highways is presented in which sensitivity to the stochastic nature of the various factors involved in the design process and utilization of their distribution are used in calculating design values. The proposed approach also attempts to achieve a cost-effective design by taking into account all the cost elements associated with the highway. An empirical example of a horizontal curve demonstrates the advantages of the probabilistic approach.

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This paper is concerned with the concepts used in the geometric design of highways. A modified approach is proposed that would achieve more meaningful and cost-effective designs.

Current geometric design practice is based heavily on design standards and the following basic design process is used. First the highway section to be designed is classified into one of the several functional classes (e.g., freeway, arterial, local). Then a design speed is selected for the highway on the basis of its classification and local conditions. After highway classification and design speed have been specified, design values for the various highway elements are selected from a set of predefined design standards *(1–3)*.

This design practice has two major advantages. First, the design concepts are transparent. This enables highway engineers to be trained easily and quickly. Second, this practice supports consistent design. For example, geometric elements of freeways in different locations designed for 100 km/hr will have the same design values.

The foregoing practice, however, has been subjected recently to increased criticism. It has become evident that such a practice is rigid and does not allow the designer to use his own judgment in special cases in which deviations from the standards are clearly justified *(4)*. The practice is not always sensitive to important factors such as traffic volume, construction cost, and traffic composition. That is, once the highway class and the design speed have been selected, the minimum design value of a horizontal curve, for example, is fixed, regardless of the traffic volume that will use the road and the costs associated with implementing the design standard. Thus, a horizontal curve on an arterial road designed for 80 km/hr and serving a very low traffic volume in mountainous terrain will have the same minimum design value as a horizontal curve on a highway that serves a high traffic volume in level terrain.

Another criticism is that inflexible geometric design standards tend to be based mainly on safety considerations, which results in excessively high design standards in many situations *(5)*. For example, in the design of a vertical curve, the relevant inputs are the driver's reaction-perception time, the speed, the friction factor, and the driver's eye height. The design standards specify safe values for all these factors. For example, a reaction-perception time of 2.5 sec is used because it is valid for a large percentage of the population. Thus, the sight distance based on these values would result in a costly design.

Still another criticism addresses the concept of the design speed, which is the basis of the current design practice. The design speed is defined as the speed of the 85th-percentile driver in the speed distribution. However, it is not always clear which distribution is applicable, because there is a tangent speed distribution, a curve speed distribution, and speed distributions for cars and trucks. A recent change in design practice has been the replacement of the design speed concept with the concept of a consistent design *(6, 7)*, which is also based on the 85th-percentile driver. In this approach it is desired to limit the maximum value of speed changes that the 85th-percentile driver experiences. However, it has been shown *(8)* that the 85th-percentile driver in the speed distribution found for a tangent highway section is not the same as that in a curve speed distribution. Moreover, it is theoretically possible for the speed distributions on a tangent section and on a following curve to be identical, which implies a consistent design, yet all drivers may experience significant speed changes.

The use of a design speed has also been criticized in other studies that claim that it is irrelevant in specific cases, especially in the case of a horizontal curve *(9, 10)*. In the latter case, the determination of the design speed value for a curve with a given radius and superelevation rate is based on the value of the maximum superelevation rate. Thus, the same horizontal curve (i.e., the same radius and superelevation rate) may have various design speed values based on various values of the maximum superelevation rate.

The foregoing deficiencies reflect the deterministic approach adopted in these design processes. Although all the factors involved in the geometric design process (e.g., speed, friction, reaction time) are stochastic in nature and are distributed among road users, the current approach is based on a single, arbitrarily chosen value to represent each factor. The failure to account for the stochastic nature of these factors is likely to lead under some circumstances to poor designs. That is, in some cases the combination of those deterministic values that are chosen out of the distributions may not be representative of the road user population, and hence may result in an unsuitable design of a highway section.

In summary the current design process is not sufficiently sensitive to traffic volume and composition and does not explicitly consider the variation of cost factors among different locations. Also, because of its deterministic nature, it may lead to a poor design. These problems have been recognized by highway engineers for some time and several solutions have been proposed. In order to overcome the excessively high construction costs that often result from adherence to general design standards, alternative design standards have been developed for special cases such as low-volume roads *(11)*, low-cost roads, and roads in developing countries *(12)*. These efforts justify the use of standards that are lower than the usual ones but continue to employ the deterministic design process that is based almost entirely on road classification and design speed.

A modified approach to geometric design of highways is presented here. This approach is fully sensitive to the specific conditions of the design problem at hand, that is, to traffic volume and composition and to construction, maintenance, and user costs. It is also based on the stochastic nature of the various factors involved in the design process. The proposed approach is an attempt to obtain an optimal or a cost-effective design that takes into account all the cost elements associated with the highway. In this approach the designer is made aware of the economic and safety implications of alternative designs.

The proposed approach is presented along with an empirical example. The differences between the current and the modified approach are demonstrated, and the potential of a future development in the proposed direction is discussed.

THE PROPOSED APPROACH

The first stage in the proposed approach is to obtain information about traffic volume, traffic composition, driver performance, and vehicle characteristics. The relevant information includes reaction-perception times, speed distributions at various highway locations, and vehicle dimensions and characteristics such as friction factors. It is recognized that precise information for a specific site may not be available. However, empirical studies may be used to obtain reasonable distributions for the relevant parameters. It may be noted that in the current design approach the designer uses empirical values determined in other studies to design new highways.

The major difference, however, between the current approach and the proposed approach is that the current approach requires a single deterministic value for each parameter and the proposed approach utilizes the full distribution of parameter values. For example, in order to determine the required design value (i.e., the standard) of sight distance for a design speed (V) of 80 km/hr, the current approach assumes that the reaction-perception time (t) is 2.5 sec, the friction-factor value (f) is 0.2, and thus the required sight distance (S) is 80 m, which results in the following expression for stopping distance:

$$S = Vt + V^2/(2gf) \tag{1}$$

where g is the gravity acceleration constant. The proposed approach assumes that the distributions of the travel speed, the reaction-perception time, and the friction factor are known. On the basis of many empirical studies, it is possible to assume these distributions when observations are not available. It may be noted that these parameters are not independently distributed. The relevant factors are often correlated with the travel speed. For example, the friction-factor value decreases with increasing travel speed, and the perception-reaction time may also.

The second stage of the proposed approach is the determination of the relevant physical or behavioral relationships, or both, from which the design value may be calculated. In the current approach, these are the various design equations that relate the design value to the design speed, as in Equation 1. As was mentioned earlier, however, more than one relationship may determine the design value of a specific element. For example, one may design a horizontal curve to satisfy the dynamic forces acting on the vehicle, and thus use the following relationship:

$$R = V^2/[g(e + f)] \tag{2}$$

where R is the curve and e is the superelevation rate. On the other hand, one may wish to have a consistent design such that

$$\mathrm{Max}(V_t - V_c) = 15 \tag{3}$$

where V_t is the travel speed on the tangent section and V_c is the speed on the curve.

The variables on the right-hand side of the various relationships will be referred to here as the input parameters. Thus, the purpose of the first stage in the proposed approach is to determine the distributions of the input parameters. Once the input parameters are available, these distributions can be incorporated into the design relationships to get an output distribution for the parameters on the left-hand side of the various relationships. In the current approach, the resultant parameter on the left-hand side is the desired design value, or the standard. In the proposed approach, however, a full distribution of possible design values is obtained, out of which only one may be selected. Thus, the variables on the left-hand side in the proposed approach are referred to here as the intermediate variables. The analytical determination of the derived distribution of an intermediate variable is not a simple task. Consider, for example, the sight-distance relationship in Equation 1. Even if all the input parameters are independent and normally distributed, the distribution of the stopping distance is not normal. If the intermediate variables were a linear function of the input parameters, the distribution would also be normal. [An example is the application to the design of a climbing lane by Ben-Akiva et al. *(13)*.] In other situations it is always possible to apply a numerical approach to determine the derived distribution.

To demonstrate the ideas presented so far and the feasibility of applying a numerical approach, the following empirical example for the design of a horizontal curve may be considered. The relevant physical relationship that governs the curve design was given in Equation 2.

In this example, the maximum superelevation rate used is 6 percent. It is assumed that the speed distribution can be approximated by the normal distribution with a mean value of 50 km/hr, and two values for the standard deviation are compared: 25 km/hr (e.g., heterogeneous traffic, which may include slow trucks and fast cars) and 10 km/hr (e.g., more homogeneous traffic). By using these two speed distributions, a sample of 1,000 vehicle speeds was randomly drawn from each distribution, and the resultant two distributions are shown in Figure 1.

The side friction factor has been shown in many studies to be highly correlated with the travel speed *(14)*. It is assumed that the side friction factor is normally distributed; that is,

$$f \sim N(\mu_f, \sigma_f^2) \tag{4}$$

On the basis of an empirical study reported recently by Lamm *(14)*, it is assumed that

$$\mu_f = 0.37 \cdot (0.0000214 \cdot V^2 - 0.0064 \cdot V + 0.77) \tag{5}$$

where V is the travel speed in kilometers per hour. The standard deviation of the distribution is assumed to be 0.0555. For each vehicle in the sample drawn from the speed distribution a friction-factor value based on its speed is now determined. A value was randomly drawn from a normal distribution with the mean value given by Equation 4 and a standard deviation of 0.0555. The resultant friction-factor distributions for the two speed distributions are presented in Figure 2.

It may be noted that the percentile values in Figure 2 indicate the percentage of the population for which the given friction factor or less is applicable. However, the geometric design process considers the percentile of the population for which at least a given value of the friction factor is applicable. Thus, a friction-factor value that covers 90 percent of the population corresponds to the 10th percentile in Figure 2.

Each vehicle in the sample has now been assigned both a travel speed and a friction factor. By using Equation 2 the minimum horizontal curve radius required by that vehicle may be calculated. Because each vehicle has a different speed and side friction factor, each vehicle also requires a different minimum curve radius. Figure 3 presents the curve radius distribution for the two samples for the two speed distributions. As may be expected, the wider speed distribution results in a wider radius distribution.

Figure 3 shows, for example, that 90 percent of the drivers for the relatively homogeneous speed distribution may be satisfied with a curve radius of 150 m, whereas a curve of 265 m is required to satisfy 90 percent of the drivers under the heterogeneous speed distribution. The corresponding radii for 85 percent of the population are 135 and 220 m, respectively.

At this point it may be useful to compare the radius distribution in Figure 3 with the design standard that may result from

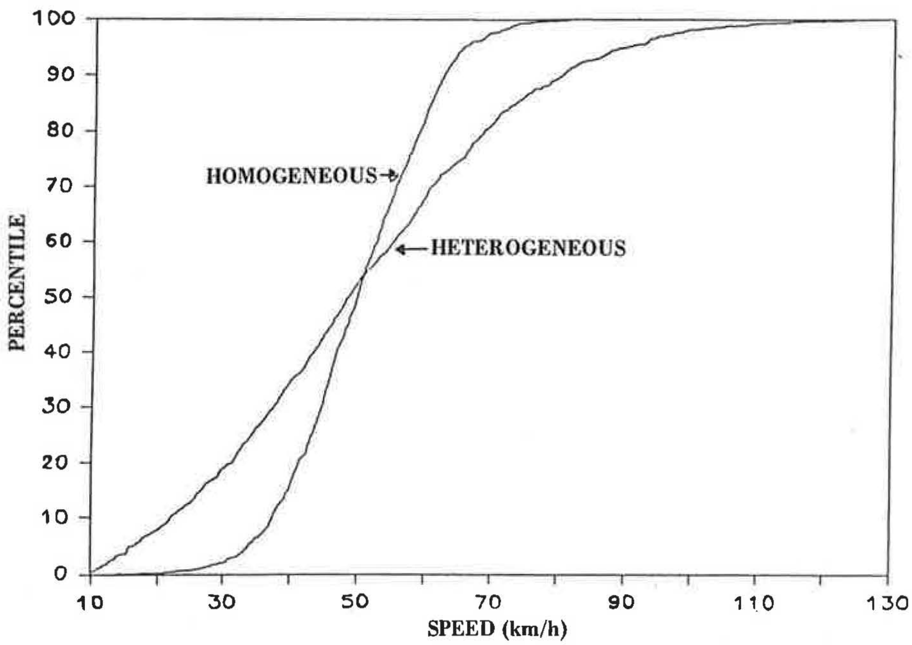

FIGURE 1 Simulated speed distribution.

the current approach. As was mentioned earlier, the design speed is usually the speed of the 85th-percentile driver. Thus, the narrow speed distribution results in a design speed (V_1) of 60.2 km/hr and the wide speed distribution results in a 73.4-km/hr design speed (V_2). In order to calculate the value of the minimal radius, a friction-factor value must be assigned to each design speed. For that purpose the relationship in Equation 4 is used, which gives the mean value of the friction factor. These values for Equation 4 are 0.17 and 0.15 for V_1 and V_2, respectively. However, for design, safe values of the friction factor are necessary rather than mean values. Because the standard deviation of the friction factor distribution is assumed to be 0.0555, the 85th percentile is approximately the mean value minus one standard deviation, and the 90th percentile is approximately equal to the mean minus 1.3 times the standard deviation. Thus, the 85th-percentile friction factors are 0.115 and 0.097 for V_1 and V_2, respectively, and the corresponding 90th-percentile values are 0.099 and 0.081 for V_1 and V_2, respectively. The 90th-percentile friction factors yield radii of 179 and 301 m for V_1 and V_2, respectively. Incorporating the design speed value and the 85th-percentile friction-factor value into Equation 2 yields curve radii of 162 and 269 m for V_1 and V_2, respectively.

With Figure 3 it can now be determined what percentile of

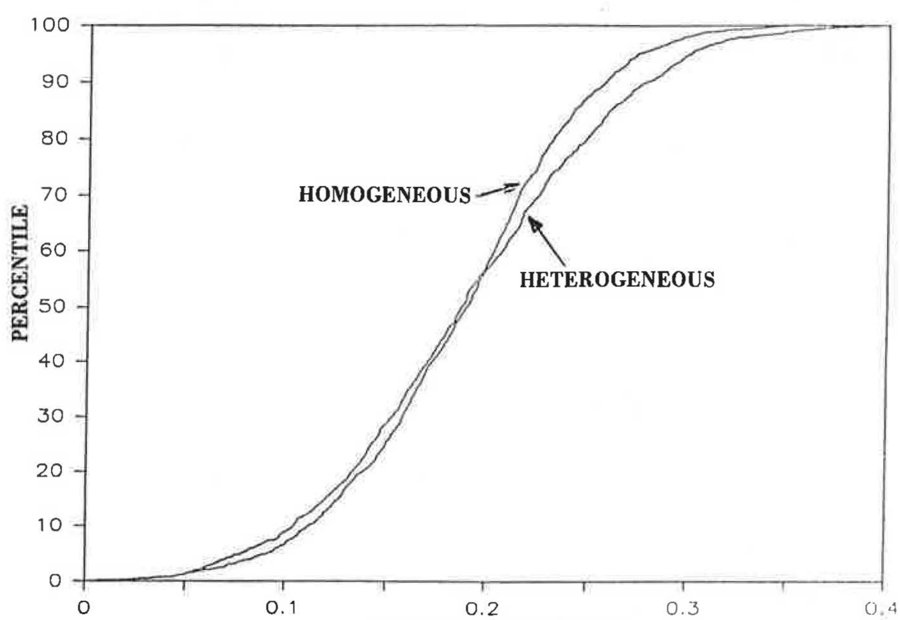

FIGURE 2 Simulated friction-factor distribution.

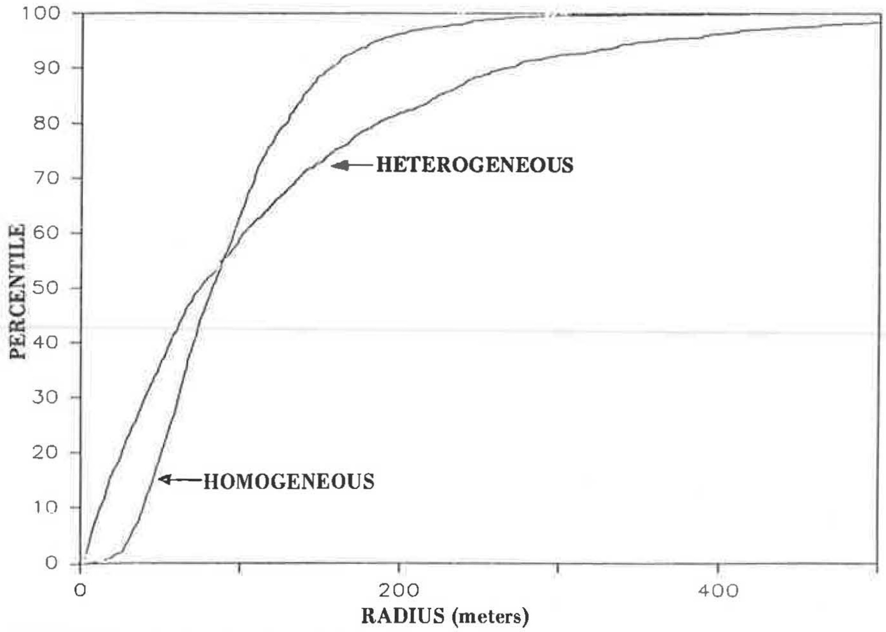

FIGURE 3 Desired radius distribution.

the road user population is satisfied by these design values. In the first case, where the 85th-percentile values of the friction factor are used, the corresponding percentiles of the population that are covered by the design standards are 93 and 90 percent for V_1 and V_2, respectively. In the second case, where the 90th-percentile values of the friction factor are used, the percentiles covered by the standards are 95 and 93 percent, respectively.

Thus, a design speed of 73 km/hr and a corresponding 85th-percentile friction factor result in a standard that covers 90 percent of the population, whereas a design speed of 60 km/hr with an 85th-percentile friction factor result in a standard that covers 93 percent of the population. The results show that using the 85th-percentile values of the speed and friction-factor distributions may allow the derivation of a design standard that covers 93 percent of the population, which may be viewed as an overdesign, because it was intended to satisfy only 85 percent of all drivers.

Once the output-value distribution, as shown in Figure 3, has been established, there is a need to select only one design value from the distribution. To do so, a percentile criterion may be used that states that the single design value selected needs to satisfy the requirements of at least a certain percentage of the drivers. As will be restated later, there is no need to state a priori the value of the minimum percentile. Examination of the shape of the output-value distribution may lead to a reasonable selection of a design value.

The percentile criterion has an intuitive safety implication in that the higher the percentile value is, the safer the design is. However, it is obvious that there is no need to design the highway for the 100th-percentile driver. On the basis of the output-value distribution, the designer may determine the implication of a specific design alternative. For example, if the curve radius is constrained to be only 100 m, the designer may determine that such a design will satisfy the requirements of only 60 percent of the drivers.

Besides the intuitive meaning, another advantage of the percentile criterion is its sensitivity to the traffic composition. As may be seen from Figures 1 and 3, more homogeneous traffic volumes (fewer vehicle types and homogeneous driver performance) result in lower design values.

The disadvantage of the percentile criterion is the lack of sensitivity to the volume of traffic and cost considerations. Thus, another design criterion, called the minimum-cost criterion, is suggested here. An objective function is defined that expresses an expected total cost as a function of the design value, and the value that brings this function to a minimum is selected. The cost function has two components: the road user cost and the construction and maintenance cost. Each possible design value results in a different road user cost and a different construction and maintenance cost, and hence a different total cost. In general, a safer standard will imply savings in user cost and increased construction cost, and a chosen design value reflects a specific trade-off between these two cost components. A cost-minimization approach has been used in past studies *(15)* to help in the selection of a cost-effective design among various discrete alternatives. The uniqueness of the criterion proposed here is that it accounts for the full range of all the possible design values and is linked with the distributions of the intermediate values. The importance of the second aspect will be explained shortly.

The user cost function includes accident costs, vehicle operating costs, and the value of the driver's time. These cost elements are dependent on the selected design value and on the distributions of the intermediate values. Accident costs, for example, may be dependent on the difference between the design value required by a driver and the selected design value. The same applies to the fuel consumption cost, which is partly dependent on the level of the speed changes that are imposed on the drivers by the chosen design value. To calculate the total cost, it is necessary to sum these values for a heterogeneous driver population.

The other component of the total cost is the construction and maintenance cost. This component depends only on the selected design value and not on the input-value distributions.

Formally, the cost criterion may be defined as follows. Let X be the design value of interest and denote by x the values of the various input and intermediate variables. Let f(x) be the probability density function of x, C(X,x) the road user cost function, and I(X) the construction and maintenance cost. The objective function may be written as

$$\min_{X} \left[\int C(X,x)f(x)d(x) + I(X) \right] \tag{6}$$

If desired, the objective function may be subjected to various design constraints.

To demonstrate the concept behind this criterion, the problem of selecting an optimal design value for a horizontal curve R is reconsidered. Let r be an intermediate variable; that is, r is the curve radius required by each driver. For simplicity, the perfectly homogeneous case, in which all road users are identical, is used and r therefore takes a single value. It may be noted that this assumption is the one used in the current design practice; that is, all road users require the same design value. In other words, in this example r may be defined as the design standard. The road user cost may be defined as

$$C(R,r) = \begin{cases} N \cdot b_1 \cdot (r - R)^2 & \text{for } R < r \\ 0 & \text{for } R \geq r \end{cases} \tag{7}$$

in which b_1 is the accident parameter cost and N is the number of vehicles using the road. For simplicity B_1 is defined as equal to $N \cdot b_1$. In this cost function, the only relevant element is assumed to be the accident cost. Thus, when the selected design value (R) is smaller than the intermediate value (r), which is needed by road users, accidents may occur. If the selected design value, however, is equal to or greater than r, the accident costs are assumed to be zero.

The construction cost function may be approximated by

$$I(R) = b_2 \cdot R^2 \tag{8}$$

Here the construction cost is also represented by a nonlinear function with a parameter b_2, and the cost is considered to be mainly right-of-way costs. The total cost function is the sum of Equations 7 and 8. It is evident that for the case of $R \geq r$, the optimal design value is $R = r$. Thus, the optimal design value is in the range $R \leq r$ and the following objective function is derived:

$$\min_{R} \left[B_1 (r - R)^2 + b_2 \cdot R^2 \right] \quad R \leq r \tag{9}$$

The optimal solution is then given by

$$R_{opt} = r/[1 + (b_2/B_1)] \tag{10}$$

The conditions in Equation 10 indicate that the optimal design value is related to the intermediate variable r by the ratio of the parameters of road user cost to construction cost. Unless b_2 equals 0 or B_1 is infinite, the optimal design value (R) will always be less than the intermediate value r. It may be recalled that r is in fact the current design standard. Thus, implicitly, the current design practice ignores the trade-off between construction and accident costs. This may be a reasonable assumption to make in level terrain, but not in mountainous terrain where the cost of earth movement is a very significant factor. However, current design practice evidently does not completely ignore this trade-off with construction cost. The deterministic values selected are high (or safe) percentile values, but do not cover the 100th-percentile driver. Thus, the selection of a specific percentile value less than 100 is an implicit recognition of this trade-off. The optimal cost criterion, however, makes the trade-off explicit.

The optimal cost criterion may also be applied numerically. In this case, it was assumed that the 1,000 vehicles sampled for the heterogeneous speed distribution make up the average daily traffic volume that traverses the curve. For the cost function whose components are given in Equations 7 and 8, the following parameter values were assumed: b_1 lies in the range of $\$0.0001/(\text{day} \cdot \text{m}^2)$ to $\$0.0004/(\text{day} \cdot \text{m}^2)$ and b_2 has a value of $\$0.05/(\text{day} \cdot \text{m}^2)$, which corresponds to a construction cost of \$200/m. The total daily cost for various design values in the range of 50 to 300 m was calculated. Figure 4 presents the total cost as a function of the design value for different b_1-values. It may be seen that as the value of b_1 increases—that is, as the importance of the accident costs increases—the value of the optimal design value also increases. It may be noted, however, that the optimal value for $b_1 = 0.0002$, for example, is only 140 m, which meets the requirements of only 70 percent of road users. These results show that it may be cost effective to reduce the standard values.

DISCUSSION OF RESULTS

A new approach to geometric design of highways has been presented that utilizes the full distribution of input parameters and attempts to achieve a cost-effective design.

However, there are several problems associated with the implementation of the approach. First, to be sensitive to local conditions, the approach needs the appropriate input-value distributions. In some cases this may call for an extensive data collection effort. Because many parameters may be correlated (e.g., speed and reaction time), such a data collection effort is not a simple task. Second, once the appropriate input-value distributions have been established, the relevant output-value distribution must be derived. Analytical derivation of the output-value distribution is always the preferred approach. However, for many highway design problems, the analytical derivation is a very complicated task and a numerical approach is suggested instead.

Another problem is the construction of an appropriate cost function. First, there is a need to identify all the relevant cost components. The conventional road user cost components are accident costs, vehicle operating costs, and the value of travel time. In some instances, some of these elements may not be relevant (e.g., the value of time). Also, even when the total cost components are known, there is still a need to assign a monetary value to such elements as accidents and value of time,

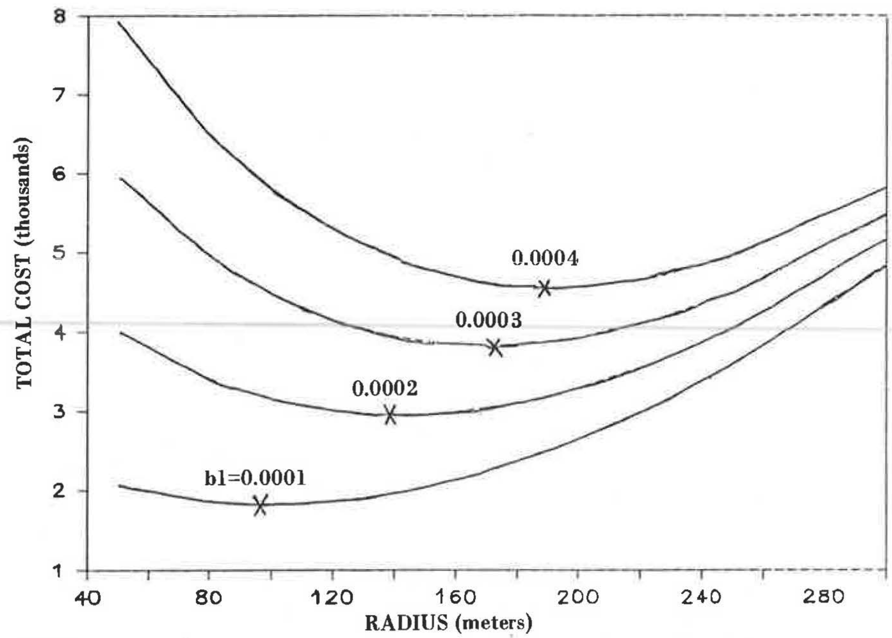

FIGURE 4 Optimal design value for a given construction and maintenance cost.

which are difficult to quantify. Because highways are designed to serve for several years, there is a need to forecast the value of the various parameters associated with the design process, such as future volumes and costs. The proposed approach involves many factors, and this may introduce additional uncertainty into the design process. As a result, the selected single design value may not be optimal or cost effective. A possible solution to this problem is to perform a sensitivity analysis by varying the values of the various parameters. An example of such an analysis has been given elsewhere *(13)* for the case of climbing-lane design.

Another methodological problem is the interdependency that may exist between the input variables and the selected design value. In the case of a horizontal curve, for example, it was found that drivers adjust their speed and the level of the accepted lateral acceleration rate to the curve radius *(16)*. The design process should be able to account for this phenomenon as well. A related issue is the three-dimensional aspect of the design. The methodology presented in this paper demonstrates the design process for a single highway element. However, highway elements are interrelated in a three-dimensional system. An optimal design should take into account all the highway elements in a given section.

The last problem discussed here is the incorporation of the new approach into practice. On a day-to-day basis, it is not practical to conduct an extensive study each time there is a need to design a highway section. Rather, it is desired to be able to upgrade the current design standards to include the features of the new approach. A simple improvement would be to introduce additional factors into the specification of a design standard. For example, the travel speed may be represented by two parameters—mean and standard deviation—and a design value may be calculated for a range of percentile values. The volume of traffic may also be introduced and a range of optimal design values may be presented for each combination of speed distribution and traffic volume.

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