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Contents

Foreword

The papers in this volume focus on cross-section and alinement design issues and were presented at the 1992, 1993, and 1994 TRB Annual Meetings. The first eighteen papers are from the sessions sponsored by the TRB Committee on Geometric Design. Of these eighteen, the first eleven focus on alinement issues, whereas the other seven deal with cross-section issues. The final five papers in this volume are from sessions sponsored by the TRB Committee on Operational Effects of Geometrics.

Easa presents geometric and sight distance characteristics of reverse horizontal and vertical curves and design values for these reverse curve elements. Lamm and Smith describe an alinement design process called curvilinear alinement, in which design element sequences are formed with specific relationships to other design elements (instead of single design elements being used more or less arbitrarily). Harwood and Mason evaluate possible skidding and rollover of passenger cars and trucks on horizontal curves with design speeds and minimum-radius curves in accordance with AASHTO policy. Choueiri et al. concentrate on low and intermediate traffic volume, two-lane rural highways, relating the effects of alinement features with accident rates. Fitzpatrick discusses the AASHTO policy on superelevation design procedures, concluding that they are based on vehicle occupant comfort and appearance, and recommends research on more rational factors on which to base the design. Lamm et al. present a case study using thematic maps to portray important environmental resources, allowing highway designers to identify corridors of low environmental conflict. In another paper Lamm et al. introduce a procedure for evaluating horizontal alinement of two-lane rural roads to identify potential safety deficiencies. Smith and Lamm discuss the proper fitting together of the horizontal and vertical roadway alinement, with a concentration on the relationship of aesthetics and safety. Kontaratos et al. analyze the relationship between the horizontal curve radius and the grade of the roadway by using an enriched model to describe the driving mode during motion of a passenger car, said to be more critical than the braking mode. Easa proposes a new unsymmetrical vertical curve, minimizing the rates of change of grades and providing a smoother ride and improved appearance. Sanchez presents an analysis of stopping sight distances along interchange ramps and connectors with combinations of minimum horizontal and vertical geometry and longitudinal traffic barriers by using 3-D models.

Movassaghi et al. discuss grade brakes where secondary roadways intersect a main highway and present methodology for analyzing and minimizing roughness to improve driver comfort and safety. Lomax and Fuhs summarize the state of the practice in the design of cross sections for high-occupancy vehicle lanes on freeways and arterial streets. Urbanik assesses the effects of factors such as safety, operations, cost, maintenance, and environmental issues on the selection of lane width and shoulder width on reconstruction projects of urban freeways. McCoy et al. present guidelines for the use of right-turn lanes along urban roadways, developed through the use of benefit-cost ratios. Gattis and Kalyanapuram evaluate street design standards in medium to large size cities in Oklahoma, comparing the standards with those in nationally recognized publications. Wooldridge relates design consistency on rural highways as measured by speed consistency and driver workload to accident records on those roadways. Martin examines legal implications of design exceptions in the context of tort liability and litigation.

Zegeer et al. relate lane and shoulder widths on low volume rural roads to accident rates. Bowman and Vecellio have written two closely related papers. The first presents the results of a study on the impacts of median type along arterial streets on the safety of vehicles and pedestrians, and the next paper focuses on the benefits of medians to pedestrians. Brydia and Pietrucha report on the determination of perception-reaction times and their relationship with Case III intersection sight distance requirements for intersection design. Streff et al. discuss the effects on traffic operations of transporting two sizes of over-wide manufactured housing units.

Design Considerations for Highway Reverse Curves

SAID M. EASA

Existing sight distance models of highway horizontal (vertical) curves are applicable only to simple circular (parabolic) curves. These simple models may greatly overestimate the lateral clearance needs for reverse horizontal curves and the curve length requirements for reverse vertical curves. No available analytical model quantifies the effects of the alignment reversal on sight distance characteristics. The geometric and sight distance characteristics of reverse horizontal and vertical curves are presented. For reverse horizontal curves the available sight distance is related to the parameters of the two circular arcs, the length of the common tangent, the locations of the driver and object, and the location of the vision-limiting obstacle. The obstacle may be located within the circular arcs or the common tangent. For reverse vertical curves the available sight distance is related to the parameters of the crest and sag arcs, the length of the common tangent, the locations of the driver and object, and their heights. The sight-hidden zone (dip) that may exist on a reverse horizontal (vertical) curve is examined. The sight distance profiles and minimum sight distances of reverse and simple curves are compared.

Sight distance is one of the basic elements in highway geometric design. The highway alignment should be designed so that the available sight distance is always equal to or greater than the required sight distance. Design values for required stopping, passing, and decision sight distances are presented by AASHTO (*1–5*), Neuman (*6*), Harwood and Glennon (*7*), and McGee (*8*). The available sight distance depends on the geometric elements of the highway. This paper addresses the geometric and sight distance characteristics of reverse horizontal and vertical curves. It will be useful first to review the previous work related to other types of highway curves and to describe the basic features of reverse curves.

PREVIOUS WORK

Besides reverse curves highway curves may be simple or compound curves. The simple curve consists of a single circular arc in horizontal alignment or a single parabolic arc (sag or crest) in vertical alignment. For simple horizontal curves, only the $S_m \leq L$ case is considered by AASHTO (*5*), where S_m is the minimum sight distance and L is the curve length. Waissi and Cleveland (*9*), on the basis of the NCHRP report by Olson et al. (*10*), addressed the $S_m > L$ case and developed approximate relationships for the available sight distance given an obstacle on the inside of the curve. The NCHRP model was extended, and exact sight distance relationships were developed (*11*). For simple vertical curves, sight distance models for sag and crest curves can be found in AASHTO (*5*) and Hickerson (*12*). Both $S_m \leq L$ and $S_m > L$ cases are considered. Sight distance on a simple sag curve with a noncentered overpass has been analyzed (*13*).

Compound horizontal curves consist of two circular arcs located on the same side of a common tangent (*12*). The lateral clearance needs on these curves have been established (*14*). Compound vertical curves (called *unsymmetrical curves*) consist of two parabolic arcs with a common tangent at the point of vertical intersection. The use of these curves may be required on certain occasions because of critical clearance and other controls (*4,5*). Sight distance models for unsymmetrical crest and sag curves were developed (*15,16*). The geometric characteristics of all types of horizontal curves and all types of vertical curves have been unified (*17,18*).

FEATURES OF REVERSE CURVES

For a given obstacle on the inside of an arc of a reverse horizontal curve, the available sight distance and lateral clearance needs could be found by using existing simple curve models. However these models will generally overestimate the lateral clearance needs because the alignment reversal reduces the needed lateral clearance. A reverse curve may exhibit a sight-hidden zone that affects traffic safety.

Reverse vertical curves are advantageous in hilly and mountainous terrains. Their use is also often necessary on interchange ramps (*19*). The geometries of reverse vertical curves (without intermediate tangents) are presented by Hickerson (*12*). The design of the crest arc of a reverse vertical curve by using simple crest curve models may greatly overestimate the required length. This is because the alignment reversal of the sag arc improves the sight distance and consequently reduces length requirements. A reverse vertical curve may exhibit a sight-hidden dip that affects traffic safety.

REVERSE HORIZONTAL CURVE

Geometric Characteristics

A reverse horizontal curve consists of two circular arcs, AB and CD, lying on the opposite sides of a common tangent (Figure 1). The radii of the two arcs are R_1 and R_2, and their central angles are I and J. The first and second tangents intersect at E, and their lengths are T_a and T_b, respectively. The intersection angle K is:

$$K = J - I \tag{1}$$

The relationships between the parameters of the reverse curve can be established by selecting arbitrary x and y coordinate axes

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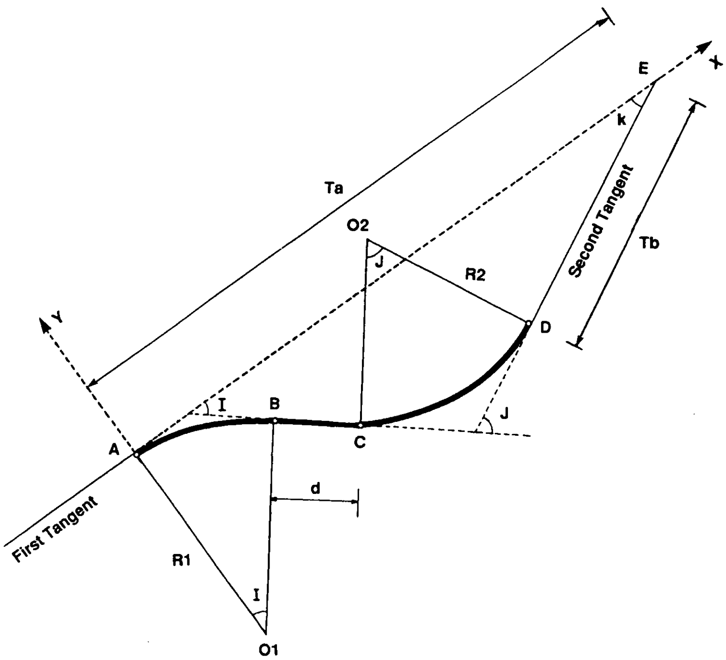

FIGURE 1 Geometry of a reverse horizontal curve.

at A, where the x-axis lies along the first tangent AE. Consider the closed traverse AO_1BCO_2DEA. Since the sum of the latitudes of the traverse must equal zero, then

$$-R_1 - d \sin I + (R_1 + R_2) \cos I - R_2 \cos K + T_b \sin K = 0 \tag{2}$$

Similarly, since the sum of the departures must equal zero, then

$$(R_1 + R_2) \sin I + d \cos I + R_2 \sin K + T_b \cos K - T_a = 0 \tag{3}$$

Equations 1 to 3 contain eight parameters: R_1, R_2, I, J, K, d, T_a, and T_b. When five of these parameters are known, including an angle, the equations can be solved to find the other three unknowns. For $K = 0$ similar relationships can easily be obtained.

Available Sight Distance

Consider an obstacle located on the inside of arc AB (Figure 2). The lateral clearance between the centerline of the inside lane and the obstacle is m_1. The angle between the obstacle and A is I_1. The available sight distance, S, depends on whether the driver is on the first tangent or on arc AB and on the location of the obstacle. There are five cases of the object location (*20*):

Case 1: Object on arc AB,
Case 2: Object on common tangent BC,
Case 3: Object on arc CD before the tangent point t,
Case 4: Object on arc CD beyond the tangent point t, and
Case 5: Object on the second tangent.

The tangent point t is the point at which the line of sight becomes tangent to arc CD. Clearly beyond this point the obstacle on arc AB has no effect and the available sight distance may then be controlled by another obstacle within the common tangent or arc CD. The obstacle may lie within the first arc or the common tangent. As an illustration, the relationships for Case 3 (with a driver on the first tangent) are derived next.

In this case the driver lies on the first tangent at a distance x from A and the object lies on arc CD before tangent point t (Figure 2). Then from triangle aAo_1,

$$\overline{ao}_1 = (x^2 + R_1^2)^{1/2} \tag{4}$$

$$I_2 = \tan^{-1}(x/R_1) \tag{5}$$

The angle at o_1 between the driver and obstacle and the distance between them, S_d, are:

$$\angle\, ao_1e = I_1 + I_2 \tag{6}$$

$$L_{11} = RI_1\pi/180 \text{ degrees} \tag{7}$$

$$S_d = L_{11} + x \tag{8}$$

The angle θ, $\overline{qo}_1$, $\overline{qf}$, and $\overline{go}_2$ are:

$$\theta = \beta - (I - I_1) \tag{9}$$

$$\overline{qo}_1 = (R_1 - m_1) \sin (180 \text{ degrees} - \beta)/\sin \theta \tag{10}$$

$$\overline{qf} = d/\tan \theta \tag{11}$$

$$\overline{go}_2 = (R_1 + R_2) - (\overline{qf} + \overline{qo}_1) \tag{12}$$

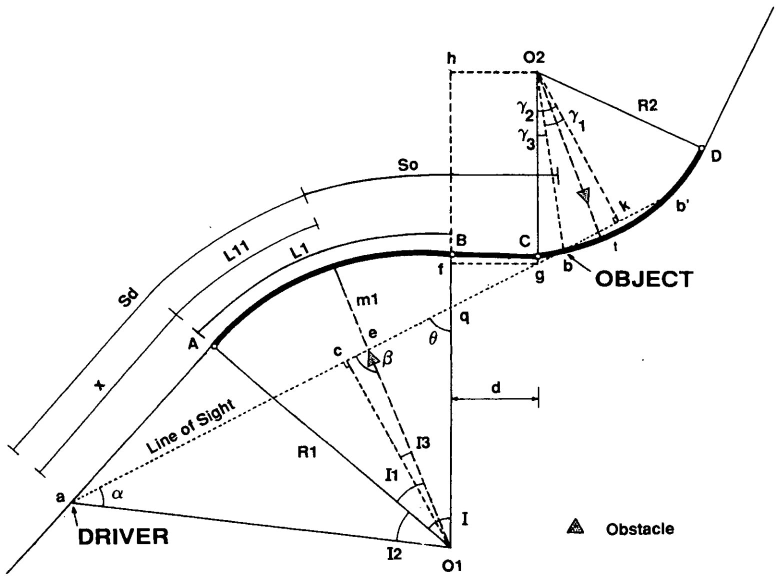

FIGURE 2 Case 3: Object on arc *CD* before tangent point *t* (example of a driver on the first tangent).

The perpendicular distance from o_2 to the line of sight, $\overline{ko_2}$, and the angles γ_1 and γ_2 are:

$$\overline{ko_2} = \overline{go_2} \sin \theta \tag{13}$$

$$\gamma_1 = \cos^{-1} (\overline{ko_2}/R_2) \tag{14}$$

$$\gamma_2 = 90 \text{ degrees} - \theta \tag{15}$$

Then γ_3 and the distance $\overline{Cb}$ are:

$$\gamma_3 = \gamma_2 - \gamma_1 \tag{16}$$

$$\overline{Cb} = R_2\gamma_3\pi/180 \text{ degrees} \tag{17}$$

The sight distance component S_o is:

$$S_o = (L_1 - L_{11}) + d + \overline{Cb} \tag{18}$$

where $L_1 = RI\pi/180$ degrees, and L_{11} is given by Equation 7. The available sight distance, S, equals $S_d + S_o$.

Finding Minimum Sight Distance

The minimum sight distance, S_m, along the reverse horizontal curve is found by computing the available sight distance, S, for successive locations of the driver until the minimum value is reached. The search starts with a large value of x and an increment, Δx. For Cases 2 and 3 the obstacle may lie below tangent *BC*, and for Cases 4 and 5 the obstacle may lie above tangent *BC*.

Practical Aspects

Sight-Hidden Zone

For Cases 1 to 3 of the object a sight-hidden zone (SHZ) is formed as shown by the zone from b to c in Figure 3(*a*). The line of sight intersects the second tangent or arc *CD* when the angle between the line of sight and the first tangent, ϕ, is greater than $-K$ (Equation 1). If K is positive (as in Figure 1) the line of sight intersects the second tangent for any location of the driver. If K is negative the line of sight intersects the second tangent when $\phi > -K$. In any case the SHZ starts when $\phi > -K$ and the intersection point c lies within the actual length of the second tangent T_2 or within arc *CD*. The length of the SHZ, L_{SHZ}, equals the distance on the road between b and c, $L_{SHZ} = S_{end} - S$, where S_{end} is the available sight distance from the driver to the end of the SHZ, and S is the available sight distance computed for Cases 1 to 3. An SHZ is undesirable because it makes the highway discontinuous and may affect the driver's perception of direction at night.

Figure 3(*b*) shows the variations of L_{SHZ} as the driver travels on the first tangent and arc *AB*. The obstacle lies within arc *AB*, where $I_1/I = 0.8$ and m_1 varies from 20 m (66 ft) to 40 m (131 ft). The driver location, X, is measured from A and is considered negative if the driver lies on the first tangent and positive if the driver lies on arc *AB*. Since K is positive (+10 degrees), the SHZ exists when the driver lies anywhere on the first tangent. For m_1 = 20 m (66 ft) the SHZ length is 273 m (896 ft) when $X = -300$ m (−984 ft) and the zone diminishes when $X = -7$ m (−23 ft). The SHZ length can be reduced (or avoided) by increasing the lateral clearance as shown in Figure 3(*b*).

Sight Distance Profile

The second arc of the reverse curve reduces the lateral clearance needs on the first arc. Figure 4 shows the sight distance profile of a reverse horizontal curve. There are two obstacles, one within arc *AB* and the other within arc *CD*, and two values of S_m, one for each obstacle. When the driver is at $X = -60$ m (−197 ft), the obstacle on arc *AB* no longer controls (SHZ diminishes). At this point the sight distance suddenly increases because it is controlled by the obstacle on arc *CD* and then gradually decreases to a second minimum value.

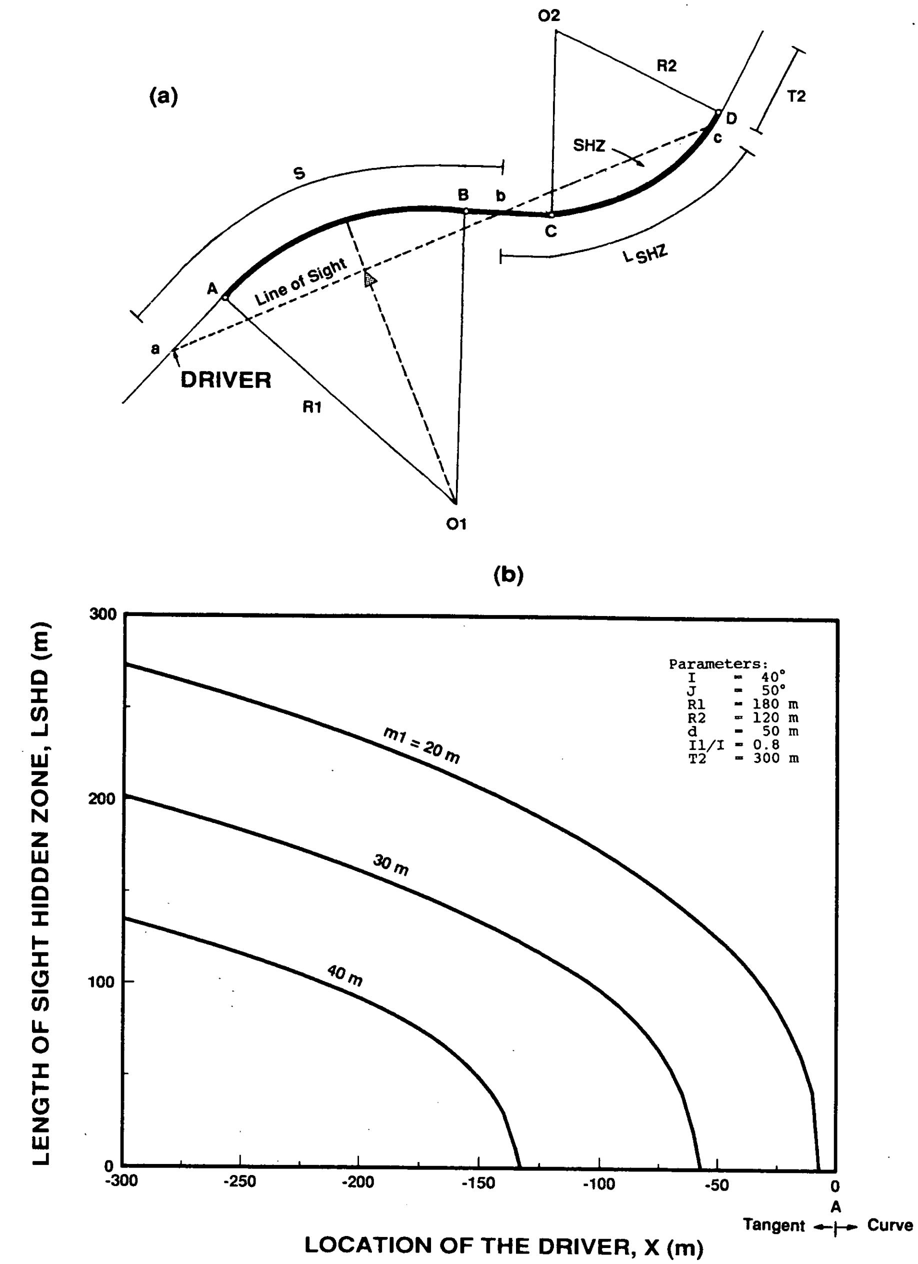

FIGURE 3 SHZ of reverse horizontal curve: (*a*) geometry of SHZ; (*b*) effect of lateral clearance m_1 on SHZ.

The sight distance profiles of a reverse curve and simple curve *AB* with m_1 = 40 m (131 ft) and I_1/I = 0.8 are shown in Figure 5. The profile of the simple curve is obtained by setting *d* equal to a very large value. S_m on arc *AB* is improved by the alignment reversal. For the reverse curve, S_m = 351 m (1,152 ft), and for the simple curve, S_m = 311 m (1,020 ft), a difference of +13 percent. The difference in S_m is large when (a) the lateral clearance is large, (b) the obstacle lies close to the common tangent, (c) the common tangent is short, and (d) R_2/R_1 is small.

Evaluation and Design Values

Table 1 gives the minimum sight distance for R_2/R_1 = 0.5, 1.0, and 100; an obstacle located within arc *AB*; and *d* = 50 m (164 ft). For R_2/R_1 = 0.5 and 1.0, the S_m values are applicable for only $J \geq 26$ and 18 degrees, respectively. For smaller *J*, S_m will be smaller than the values shown. The minimum sight distances for R_2/R_1 = 100 are about the same as those for a simple curve. The value of S_m becomes larger as R_2/R_1 becomes smaller, as expected. It is assumed that no obstacle exists on arc *CD*. If there are two obstacles, one within each arc, S_m can be found for each obstacle separately (assuming that the other obstacle does not exist). Then S_m on the reverse curve is the lesser of the two values. Note that for certain parameter values the sight distance is unlimited. In this case the obstacle is located such that the line of sight does not intersect the reverse curve when the driver lies on arc *AB* or on the first tangent within 500 m (1,640 ft) from *A*.

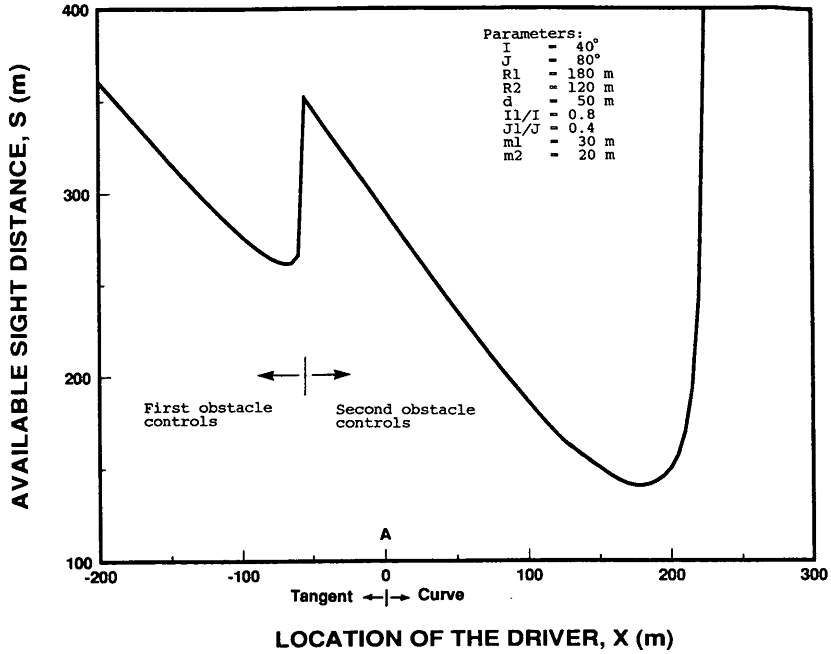

FIGURE 4 **Sight distance profile of reverse horizontal curve.**

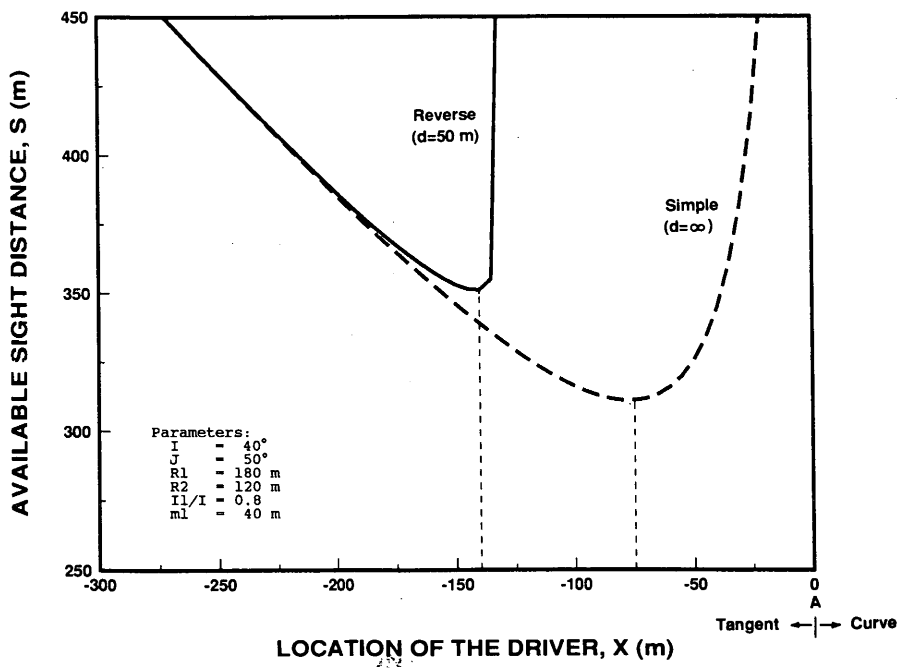

FIGURE 5 **Comparison of sight distance profiles of reverse and simple horizontal curves (obstacle on first arc).**

TABLE 1 Minimum Sight Distance on Reverse Horizontal Curve with Obstacle on First Arc (d = 50 m)

R2/R1	Cent. Angle I	Obst. Ratio (I1/I)	Radius of the First Arc, R1 (m): 100			200			300		
			m[a]=20	25	30	20	25	30	20	25	30
0.5	20	0.0	413	u[b]	u	330	504	u	336	438	605
		0.5	442	u	u	316	544	u	302	422	649
		1.0	u	u	u	u	u	u	598	u	u
	30	0.0	201	258	349	244	286	335	287	327	369
		0.5	184	245	349	206	249	304	232	271	315
		1.0	234	354	u	283	381	543	329	419	545
	40	0.0	175	203	236	231	261	291	283	316	347
		0.5	151	181	216	186	215	244	220	249	279
		1.0	180	223	281	243	289	344	300	349	404
1.0	20	0.0	352	u	u	316	434	649	332	415	526
		0.5	357	u	u	295	430	u	294	384	513
		1.0	577	u	u	444	u	u	434	658	u
	30	0.0	201	251	320	244	285	330	287	327	368
		0.5	183	236	311	206	248	296	232	271	313
		1.0	220	300	430	266	332	417	309	372	445
	40	0.0	175	203	234	231	261	291	283	316	347
		0.5	151	181	214	186	215	244	220	249	279
		1.0	178	216	260	238	277	320	293	334	377
100	20	0.0	262	320	378	293	351	409	324	382	441
		0.5	248	307	366	265	323	382	282	340	398
		1.0	262	320	379	294	352	410	325	383	442
	30	0.0	199	237	275	244	283	322	287	327	367
		0.5	180	219	258	206	245	284	232	271	310
		1.0	199	237	276	245	284	323	287	327	367
	40	0.0	175	203	231	231	261	291	283	316	347
		0.5	151	180	210	186	215	244	220	249	279
		1.0	175	203	232	231	262	291	283	316	347

a Lateral clearance in meters.
b Unlimited sight distance is available within 500 m from the start of the first arc.
Note: Minimum sight distances are expressed in meters.

For example find the required lateral clearance on a reverse curve with $R_1 = R_2 = 300$ m, $I = 20$ degrees, and $I_1/I = 1.0$ to satisfy a passing sight distance of 458 m (1,500 ft). From Table 1 $S_m = 434$ m (1,424 ft) and 658 m (2,159 ft) for $m = 20$ m (66 ft) and 25 m (82 ft), respectively. By interpolation the required lateral clearance is about 21 m (69 ft). By comparison the required lateral clearance for a simple curve is about 31 m (102 ft), as noted in Table 1 for $R_2/R_1 = 100$.

REVERSE VERTICAL CURVE

Geometric Characteristics

A general reverse vertical curve is shown in Figure 6. The curve consists of two parabolic arcs, *CD* and *EF*, with lengths L_1 and L_2 and algebraic differences in grade A_1 and A_2, respectively. The rates of change of grades are r_1 and r_2, respectively. The arcs are separated by a tangent distance d. The grades of the first and second tangents are g_1 and g_2, respectively. The grade is considered positive if it is upward to the right and negative if it is downward to the right. The grade of the common tangent *DE* is g_c. For $d = 0$, *D* and *E* coincide resulting in a point of reverse curvature. The algebraic difference in grade of the reverse curve, $A = g_2 - g_1$, is

$$A = A_1 + A_2 \qquad (19)$$

Note that A_1 and r_1 of the crest arc are negative and A_2 and r_2 of the sag arc are positive. It is desirable to design reverse curves with intermediate tangents to provide a separation between the reverse curvatures of the two parabolic arcs (*21*). In Figure 6 the two arcs have the same slope at *D* and *E*,

$$g_1 + r_1L_1 = g_2 - r_2L_2 \qquad (20)$$

The elevations of *D* and *E* and their relationship are:

$$\text{Elev}_D = g_1L_1 + r_1L_1^2/2 \qquad (21)$$

$$\text{Elev}_E = \text{Elev}_F - g_2L_2 + r_2L_2^2/2 \qquad (22)$$

$$\text{Elev}_E = \text{Elev}_D + d(g_1 + r_1L_1) \qquad (23)$$

Substituting for Elev_D and Elev_E from Equations 21 and 22 in Equation 23 and noting that $A = g_2 - g_1$, the following basic relationships for r_1 and r_2 are obtained:

$$r_1 = (AL_2 - 2W - 2dg_1)/L_1(L + d) \qquad (24)$$

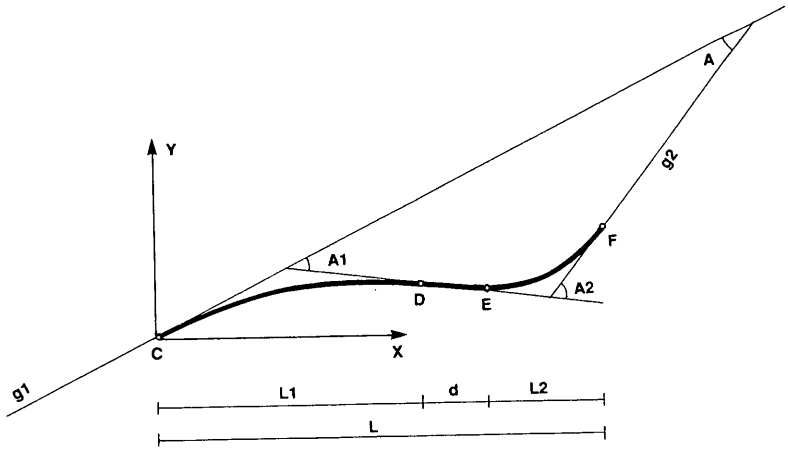

FIGURE 6 Geometry of reverse vertical curve.

$$r_2 = (AL_1 + 2W + 2dg_2)/L_2(L + d) \tag{25}$$

where

$$L = L_1 + L_2 + d \tag{26}$$

$$W = g_1L_1 + g_2L_2 - \text{Elev}_F \tag{27}$$

The sign of r_1 or r_2 will be positive if the respective arc is crest and negative if it is sag. For parallel tangents ($g_1 = g_2$) set $A = 0$ in Equations 24 and 25. It is interesting to note that Equations 24 and 25 are also applicable to symmetrical and unsymmetrical curves when W and d equal zero (for symmetrical curves $L_1 = L_2 = L/2$). The elevation of various points on the curve can easily be obtained.

Available Sight Distance

The available sight distance on the reverse curve, S, depends on the direction of travel. The relationships for S are developed for traveling from the crest to the sag arc. The relationships are also applicable to the other travel direction by exchanging the driver's eye height h_1 and the object height h_2. The available sight distance is considered for the crest curve (daytime conditions). There are two cases for the location of the driver: driver on the first tangent (Case A) and driver on the crest arc (Case B). For each of these locations there are four cases of the object location (*20*):

Case 1: Object on the crest arc,
Case 2: Object on the common tangent,
Case 3: Object on the sag arc, and
Case 4: Object on the second tangent.

The geometry of the available sight distance for these cases is shown in Figure 7. The letters in circles refer to the driver location, and the numbers in circles refer to the object location. In all cases the available sight distance, S, equals $S_d + S_o$, where S_d and S_o are the distances from the driver and object, respectively, to the tangent point a. S_d depends on the two cases of the driver, and S_o depends on the four cases of the object.

Case A: Driver on First Tangent

In Case A the driver lies on the first tangent at a distance T from the start of the crest arc. On the basis of the property of a parabola

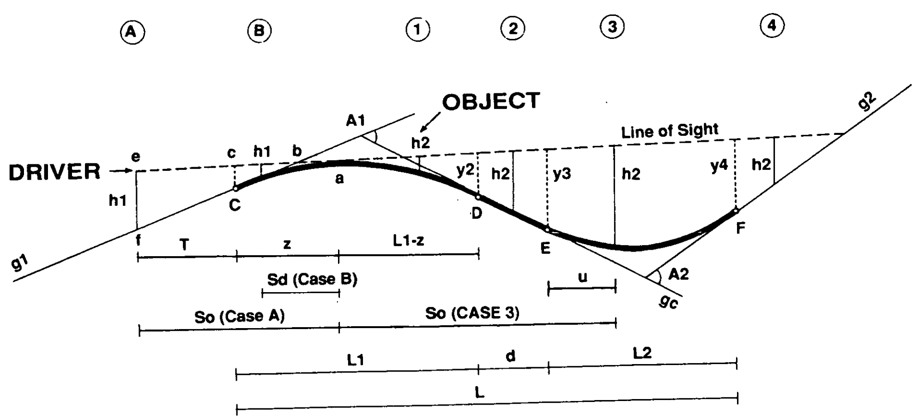

FIGURE 7 Geometry of sight distance on crest arc of reverse vertical curve.

and the similarity of triangles *bcC* and *bef*,

$$cC = -r_1 z^2/2 \tag{28}$$

$$z = T + [T^2 + (-2 h_1/r_1)]^{1/2} \tag{29}$$

where T is negative when the driver lies on the first tangent and positive when the driver lies on the crest arc. The sight distance component S_d equals $z - T$.

Case B: Driver on Crest Arc

In Case B the driver lies on the crest arc at a distance T from C. On the basis of the property of a parabola the distances v and z are (Figure 7)

$$v = (-2h_1/r_1)^{1/2} \tag{30}$$

$$z = T + (-2h_1/r_1)^{1/2} \tag{31}$$

The sight distance component S_d equals $z - T$. The sight distance component S_o depends on the object location. As an illustration the relationships for Case 3 are derived next.

Derivation of S_o *for Case 3*

In Case 3 the object lies on the sag arc and the driver lies on the first tangent or the crest arc. The vertical distances y_2 and y_3 are (Figure 7)

$$y_2 = -r_1(L_1 - z)^2/2 \tag{32}$$

$$y_3 = y_2 - r_1(L_1 - z)d \tag{33}$$

The vertical distance between the line of sight and the common tangent is

$$(h_2 + r_2u^2/2) = y_3 - r_1(L_1 - z)u \tag{34}$$

Equation 34 is quadratic in u, and its solution is

$$u_1 = [-r_1(L_1 - z) - G^{1/2}]/r_2 \tag{35}$$

$$u_2 = [-r_1(L_1 - z) + G^{1/2}]/r_2 \tag{36}$$

$$G = r_1^2(L_1 - z)^2 - 2r_2(h_2 - y_3) \tag{37}$$

If both roots of Equations 35 and 36 are positive the object can be seen by the driver at two locations on the sag curve. Between these locations a dip hidden from the driver's sight exists. For this reason only the smaller value of u is considered in computing the available sight distance. Thus the sight distance component S_o equals $L_1 - z + d + u_1$, where u_1 is given by Equation 35.

Finding Minimum Sight Distance

The minimum sight distance is found by exhaustive search. The search starts with a large (negative) value of T and an increment ΔT. For each value the available sight distance S is compared with the value of the previous iteration, S'. The search continues until $S > S'$ and then $S_m = S'$. For each iteration the cases of the driver and object are found by comparing h_2 with y_2, y_3, and y_4. The available sight distance for locations beyond the location of S_m is also obtained. This information is used to plot the sight distance profile on the reverse curve. The minimum sight distance varies with the travel direction.

Practical Aspects

Sight-Hidden Dip

A sight-hidden dip (SHD) is a safety concern on two-lane highways when a sag curve follows a crest curve [Figure 8(*a*)]. An SHD is defined as the portion of the road ahead of the driver within which an opposing vehicle will be hidden from the driver's view (*22*). In Figure 8(*a*), for a driver traveling on the first tangent with an eye height h_1, the line of sight is tangent to the crest curve at *a*. The hidden dip extends from *a* to *b*. However considering an opposing vehicle with a roof height h_2, the SHD extends from *e* to *f*. At these points the distance between the pavement and the line of sight is exactly h_2. Within the SHD this distance is greater than h_2, and outside the SHD this distance is less than h_2. Therefore an opposing vehicle within *ae* or *fb* will be visible to the driver.

To avoid the SHD the shaded area in Figure 8(*a*) should diminish so that a vehicle in the hidden dip is always visible to the driver. Practically, however, a smaller height h_2' should be used so that a portion of the top of the opposing vehicle is visible to the driver when it is at the lowest point of the dip [$h_2' = (1 - f_v)h_2$, where f_v is a visibility factor]. This is especially important for flat sag curves, where the increase in the visible portion of the opposing vehicle as it travels is small.

Figure 8(*b*) shows the effects on the SHD width of the algebraic difference in grades. The limits of each curve represent the roadway locations where the SHD starts and ends (SHD range). For example the SHD for A_2 = 6 percent starts when the driver lies on the first tangent at T = −19 m (−62 ft) and ends when the driver lies on the crest arc at T = +6 m (+20 ft). Therefore the SHD range is 25 m (82 ft). The SHD width at the start is 242 m (794 ft), and at the end the width is zero. By decreasing A_2 to 4 percent the SHD range is reduced to 7 m (23 ft). In fact for A_2 = 2 percent the SHD does not exist.

The results show that the AASHTO minimum rates of vertical curvature, on the basis of the SSD for the crest and sag curves, produce an SHD range as large as 1 km (0.62 mi). For small A_2 and small design speeds, however, the SHD range is negligible or does not exist. The AASHTO minimum rates of vertical curvature on the basis of the PSD for the crest curve and the SSD for the sag curve present no safety concerns.

Sight Distance Profile

The sight distance profile and minimum sight distance of a simple crest curve can be obtained by setting d equal to a very large value. The sight distance profiles of reverse and simple crest curves are shown in Figure 9. The reverse curve improves the minimum sight distance because the reversal elevates the driver

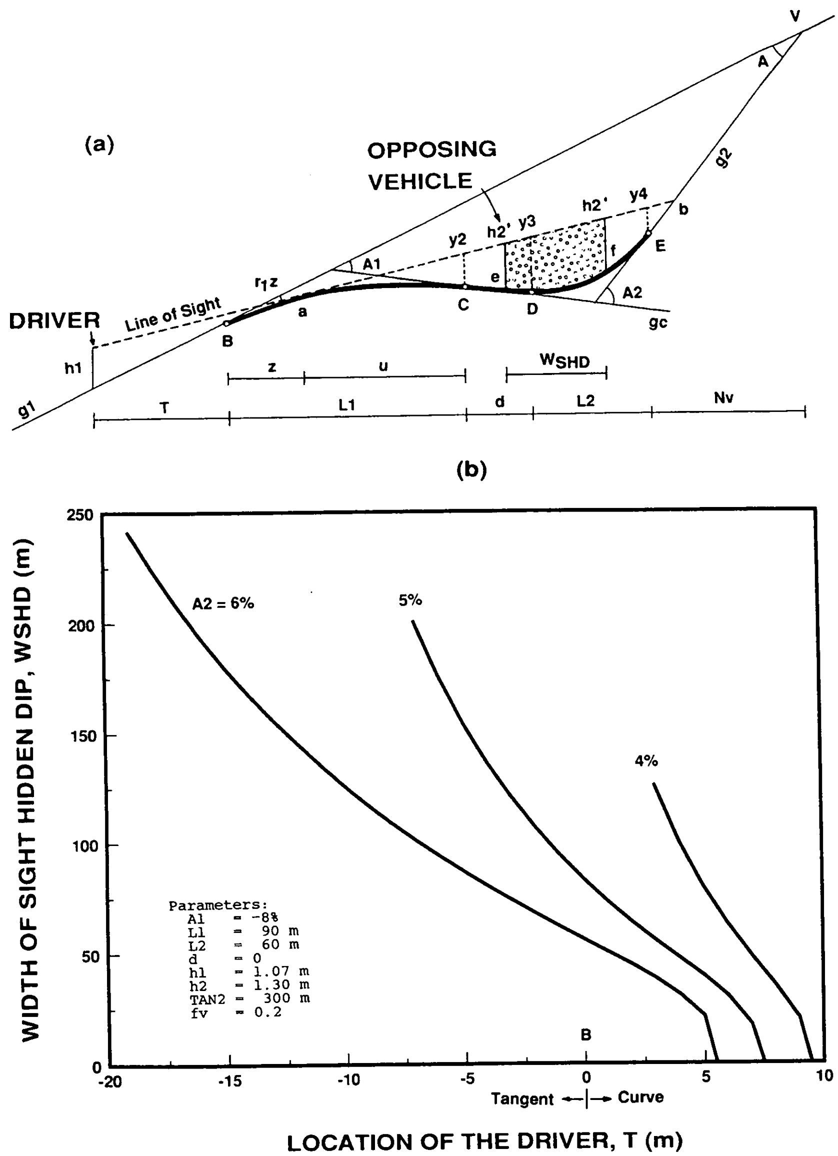

FIGURE 8 SHD of reverse vertical curve: (*a*) geometry of SHD; (*b*) effect of algebraic difference in grades, A_2.

or object. As the algebraic difference in grades of the sag curve increases, S_m increases, as expected.

The sight distance profiles of reverse and crest curves are quite different. For the reverse curve (A_2 = 16 percent) S_m occurs when the driver lies at $T = -33$ m (-108 ft). At $T = -29$ m (-95 ft) the sight distance is unlimited because the object lies above the line of sight. For a simple crest curve, however, the sight distance becomes unlimited only when the driver lies on the crest curve at $T = +15$ m ($+49$ ft) (not shown in Figure 9). The considerable difference in the shape of the sight distance profiles of reverse and crest curves affects the cost-effectiveness analysis of sites with restricted sight distances (*23,24*). For example if the required sight distance for the crest curve is 130 m (427 ft), crest curve models would predict that the length of the road with restricted sight distance is 35 m (118 ft), as shown in Figure 9. However the corresponding length on the reverse curve is only 8 m (26 ft).

S_m *Comparison with Simple Curves*

For SSD, the difference between the S_m of reverse and simple curves is large when L_1 is small and A_2 is large. The sight distance provided by a reverse curve is unlimited for small L_1. For PSD S_m is unlimited for a wider range of roadway parameters.

Length Requirements of Crest Arc

The length requirements of the crest arc of a reverse vertical curve were examined for $d = 0$ on the basis of the SSD and PSD needs of AASHTO (*5*). The lengths of the sag arc correspond to the minimum rates of vertical curvature for headlight control (upper range) on the basis of the SSD.

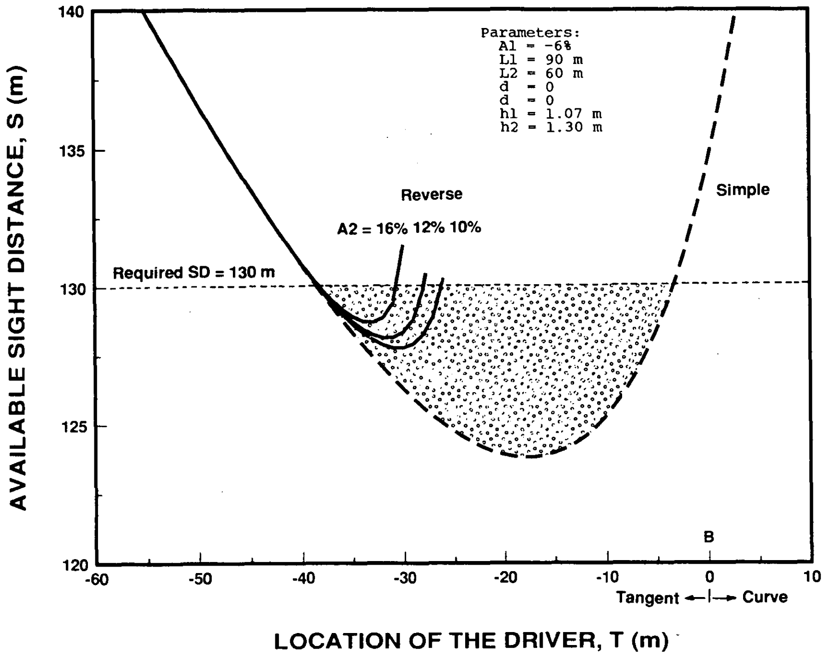

FIGURE 9 Sight distance (SD) profiles of reverse and simple crest curves.

The length requirements of the crest arc of a reverse curve on the basis of SSD are generally the same as those of a simple crest curve. The length requirements of the crest arc decrease only for limited conditions, for example, when $A_2 = -2$ percent and the design speed (DS) = 80 km/hr (50 mph). The length requirements on the basis of PSD of AASHTO are reduced to the minimum length criterion for many combinations of parameters.

What is interesting in the results is that for some cases the length requirements on the basis of PSD are less than those on the basis of SSD. This occurs, for example, for $A_1 = -2$ percent and DS = 80 km/hr (50 mph). In this case the length requirements on the basis of SSD should obviously be used, and PSD will consequently be available.

Computer Program

A computer program for the analysis of reverse horizontal and vertical curves has been developed. The program, which operates on a UNIX computer system, plots a sight distance profile, finds S_m along the reverse curve, and provides the characteristics of an SHZ or SHD. At present the program analyzes one set of curves at a time. The program will be modified so that an entire highway segment can be analyzed and sections with restricted sight distances are flagged. For reverse horizontal curves the program does not handle transition curves that affect sight distance. For simple circular curves transition curves have been found to reduce the required lateral clearance by a maximum value of 1.22 m (4 ft) (*10*).

SUMMARY AND CONCLUSIONS

No analytical method for evaluating the sight distance on highway reverse curves is available. This paper presented the geometric characteristics and available sight distances of reverse horizontal and vertical curves. Procedures for finding the minimum sight distance and analyzing the SHZ and SHD were addressed.

For reverse horizontal curves the SHZ length is related to the curve and obstacle parameters and to the length of the first and second tangents. Thus the analyst can examine the effects of different factors on the SHZ and find the lateral clearance required for avoiding the SHZ. The lateral clearance needs of reverse horizontal curves are generally less than those of comparable simple curves. This is due to the alignment reversal that improves the sight distance in comparison with that for a continuous tangent.

For reverse vertical curves the alignment reversal improves the sight distance and consequently reduces the required length of the crest arc. The results indicate that the length requirements of the crest arc are considerably less than those of a simple crest curve for certain ranges of roadway parameters. What is interesting is that, for certain combinations of parameters, the sight distance is unlimited and only a length equal to the minimum length criterion of vertical curves is needed.

This paper was concerned with the sight distance characteristics on the crest arc of a reverse vertical curve. The sight distance on the sag arc (nighttime conditions), which is also affected by the presence of the crest arc, needs to be explored. Another interesting area for future research is sight distance in three dimensions, which occurs when both horizontal and vertical curves exist at the highway location.

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Curvilinear Alinement: An Important Issue for More Consistent and Safer Road Characteristic

RUEDIGER LAMM AND BOB L. SMITH

A highway alinement design process called *curvilinear alinement* is described. It is based on a process called *relation design*, which means that no more single design elements with minimum or maximum limiting values are put together more or less arbitrarily; rather, design element sequences are formed in which the design elements following one another are subject to specific relations or relation ranges. Quantitative criteria are given for evaluating the driving behaviors of motorists in the transitions between successive design elements as well as tuning the operating speed to the design speed for single design elements on two-lane rural roads. The curvilinear alinement can provide sounder, more consistent road alinements. The suggested procedure for modern highway design provides better quantifiable and more sophisticated criteria than those that already exist in western European design guidelines. It is recommended that curvilinear alinement design be evaluated for inclusion as a recommended design process in the AASHTO Green Book for two-lane rural roads.

Each year more than 500,000 people are killed in vehicle accidents—or about one life every 2.5 min—and more than 10 million people are injured worldwide. Of the millions who are injured, tens of thousands are maimed for life. The financial costs are many thousands of millions of dollars annually (*1*). Road accidents are now the main cause of death for young people in the 15- to 25-year-old age group (*2*).

It is estimated that more than 60 percent of the fatalities can be attributed to accidents that occur on two-lane rural highways, and at least half of them can be attributed to those that occur on curved roadway sections (*2*). Thus, it becomes understandable that curved sites and the corresponding transition sections represent the most critical locations when considering measures for reducing accident frequency and severity.

It is the purpose of this paper to describe a practical design procedure, expressed here by the term *curvilinear alinement*, to help alleviate the above-mentioned problems. This procedure is developed for the practical design and possible redesign of two-lane rural roads, because multilane highways are much safer.

BACKGROUNDS, INTERIM RELATIONS, AND GOALS

The background for the study was the call for papers by TRB for the conference session Cross Section and Alinement Design Issues (1993). According to this call the task should be to address the most recent AASHTO Geometric Design policy to discuss implications and consequences, to present findings or solutions, and to include recommendations or suggestions for consideration in future editions of the AASHTO Green Book (*3*) to incorporate current practice, experience, and research.

The authors selected the subject curvilinear alinement to illustrate the positive safety impacts that may be accomplished by establishing an appropriate design procedure for two-lane rural roads. An important goal was to present important findings for future editions of the AASHTO Green Book (*3*).

To achieve this important goal it was necessary to refer to theoretical and practical basic research, which was established since 1986 for the United States. In that year a paper, "Comparison of Different Procedures for Evaluating Speed Consistency" was presented by TRB (*4*); that paper included the methods of Leisch and Leisch (*5*), the Swiss (*6*), and the Germans (*7*). It was followed in 1988 by "Possible Design Procedure to Promote Design Consistency in Highway Geometric Design on Two-Lane Rural Roads" (*8*); this first attempt at a proposal on design was based on the actual driving behaviors and accident situations for 322 curved roadway sections in the state of New York. That paper (*8*) was based mainly on the safety criterion of achieving consistency in horizontal alinement.

Meanwhile, Safety Criterion I for evaluating good, fair, and poor design practices was further developed and completed. As shown in Figure 1 the classification is based on the following:

1. The experience of Criterion I, that the driving behaviors of motorists expressed by the absolute difference of the 85th percentile speeds between successive design elements (for example, tangent to curve or curve to curve) should fall into certain ranges when evaluating good, fair, and poor design practices; and
2. The experience of Safety Criterion II, that, considering single design elements alone (for example, tangents or curves), the absolute difference between the observed 85th percentile speed and the design speed should also correspond to certain ranges.

For proposed Safety Criteria I and II, the analytical background was developed on the basis of multiple regression analysis to be able to describe the relationships between design parameters on the one hand and operating speed and accident risk (rate) on the other hand under real-world conditions (*9–12*). In doing this, recommendations regarding consistency in horizontal alinement or achieving a more consistent road characteristic could be given according to the criteria of Figure 1. Also see the section Suggested Procedure for Modern Highway Design later in this paper.

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SAFETY CRITERION	GOOD	FAIR	POOR
	DESIGN PRACTICES		
I	$\lvert V85_i - V85_{i+1} \rvert \leq 10$ km/h	10 km/h < $\lvert V85_i - V85_{i+1} \rvert \leq$ 20km/h	20km/h < $\lvert V85_i - V85_{i+1} \rvert$
II	$\lvert V85 - V_d \rvert \leq$ 10km/h	10km/h < $\lvert V85 - V_d \rvert \leq$ 20km/h	20km/h < $\lvert V85 - V_d \rvert$

V85 = 85th Percentile Speed ; V_d = Design Speed

FIGURE 1 Ranges of safety criteria for good, fair, and poor design practices.

The practical fundamentals elaborated in this paper are extended in another paper by Lamm and Guenther, in this Record; that paper is based on a case study in which an old, unsafe alinement was transformed into a sound curvilinear one by introducing complex data processing systems. Although the superior goal of the considerations so far was traffic safety, other superior goals, such as esthetics, environment, function, traffic quality (capacity), and economy, are also of great importance.

Esthetics is discussed by Smith and Lamm in another paper in this Record, and environment is presented by Lamm and Guenther in another paper in this Record.

HISTORICAL DEVELOPMENT OF ALINEMENT DESIGN PROCEDURES

Alinement design procedures are influenced primarily by the experience and education of the highway design engineer. The development started with simple polygonal sections that described the horizontal alinements, which were then based on circular curves. Finally, alinements were developed by using the standard elements tangent (straight), transition curve (clothoid or spiral), and circular curve in the horizontal alinement and the elements tangent, circular curve, and quadratic or cubic parabola in the vertical alinement. Generally, early incorporation of the vertical alinement into highway geometric design and mutual tuning with the horizontal alinement are adopted today.

Figure 2 shows the development, over time, of alinement design:

1. Tangent and circular curve.
2. Tangent and circular curve with transition curve (circular curve with double radii of curve as transition curve).
3. Tangent and circular curve with transition curve (clothoid or spiral, cubic parabola, etc.).
4. Alinement as for item 3, but without any interim tangent.
5. Three-dimensional alinement with superimposed distortion points as in item 4, but including the vertical alinement. This could be called an *ideal curvilinear alinement* (*13,14*).

It follows that the exact evaluation of the road characteristic is one important step in designing consistent and understandable curvilinear roadway sections. In this connection two-lane rural road safety is an issue of pressing national concern in Europe and the United States. These roads have the highest accident rates of any class of highway, with fatalities and injuries per vehicle kilometer of exposure (accident rates) consistently four to seven times higher than those on rural interstate highways (*15*).

Although design speed has been used for several decades to determine allowable horizontal alinement, it is possible to design certain inconsistencies into highway alinement, especially on two-lane rural roads. At low and intermediate design speeds, the portions of relatively flat alinement interspersed between the controlling curvilinear portions may produce operating speed profiles that may exceed the design speed in the controlling sections by substantial amounts (*5,8–11,13*). This is true for transition sections between successive design elements (Safety Criterion I) and for the observed single design element (Safety Criterion II) (Figure 1).

To overcome this weakness in current practice, consideration of curvilinear alinement becomes of significant importance.

Multilane highways, on the other hand, are much safer. For example, the U.S. Interstate system, with 8.7 percent of the total number of fatalities, and the comparable German *Autobahn* system, with about 9 percent of the total number of fatalities, represent the safest road classes, even though 25 percent of the vehicle kilometers driven are normally done on these roads (*2*). Thus, multilane highways and freeways are normally designed very generously. That means that curvilinear aspects are more or less included in the design of those roads in the United States and western Europe. Therefore, the following procedure primarily concerns two-lane rural roads.

CURVILINEAR ALINEMENT

In connection with a consistent road characteristic, consideration of curvilinear alinement becomes of significant importance.

U.S. Practice

The term *curvilinear alinement* in the United States is usually considered to mean a long-curve, short-tangent type of alinement, as opposed to the more common long-tangent, short-curve type of alinement. Furthermore, curvilinear alinement and the coordination of horizontal and vertical alinement are recognized as techniques for achieving an esthetically pleasing three-dimensional highway alinement.

Thus curvilinear alinement in the United States has principally been seen as a tool for achieving highway esthetics rather than as a tool for specifically achieving increased highway safety.

The 1990 Green Book (*3*) recommends the following:

- All of the pertinent features of the highway should be related to the design speed to obtain a balanced design.
- Changes in design speed should be in increments of no greater than 16 km/hr (10 mph).
- The use of greater sight distances or flatter horizontal curves is encouraged.
- Winding alinement composed of short curves should be avoided because it usually is a cause of erratic operation.
- In an alinement predicated on a given design speed, use of maximum curvature for that speed should be avoided whenever possible. The designer should attempt to use generally flat curves, retaining the maximum for the most critical conditions.

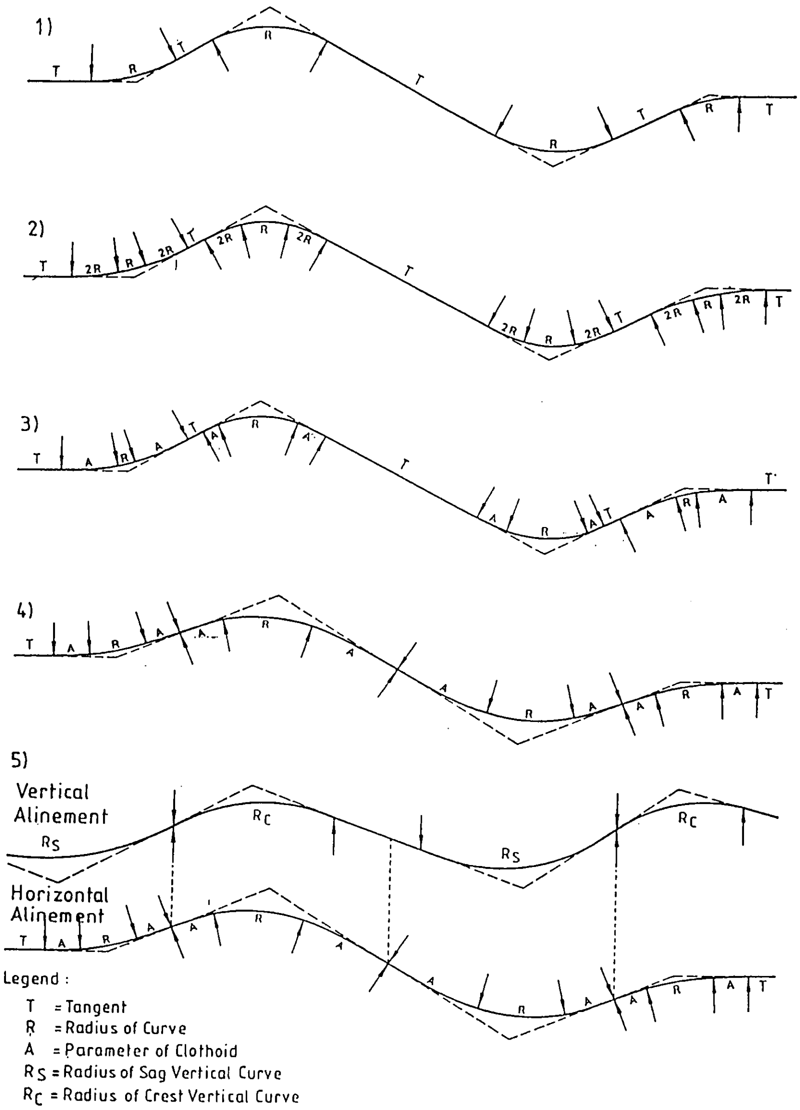

FIGURE 2 Development of alinement design (*13*).

• Consistent alinement should always be sought. Sharp curves should not be introduced at the ends of long tangents. Sudden changes from areas of flat curvature should be avoided.

With regard to human factors, H. Lunenfeld (*16*) made the following statements at the 1992 TRB meeting; these statements should be carefully considered in every highway geometric design guideline:

> Consistency is an often-violated aspect of geometric design. Driver expectancies are developed through experience and knowledge gained by driving a facility and is directly related to the geometric consistency of that facility. Consistency affects how drivers perceive and react to the information provided, by means of signing and pavement markings. Geometric consistency reinforces driver expectancies, which aids the driver in making quick and correct responses to decisions. (*16*)

In addition he warned, "Incompatibility in geometric and operational requirements may be caused by trying to fit together geometric components conveniently and economically rather than trying to satisfy operational requirements. Therefore, design consistency should be maintained," and one consequence would be a more curvilinear alinement, besides standardizing, additionally, "roadside features such as concrete barrier walls, aluminum guardrail, signing, pavement marking, traffic control devices and the like."

In conclusion, in the United States an acceptable design is one in which each design element, such as radius or degree of curve, superelevation rates, vertical curves, and sight distances, meets the Green Book (*3*) minimum or maximum requirements for the individual design element for the selected design speed(s). No specific guidelines are given in the Green Book for relationships among design elements that occur in sequence; that is, no mention is made of nor is guidance given on any design procedure comparable to curvilinear alinement design for achieving better consistency and safety on two-lane rural roads.

German Practice

Besides the Swiss (*6*) and the Swedish Guidelines (*17*), recommendations about consistency, and thereby curvilinear alinement, are found in the German Guidelines for the Design of Roads (*7,18*).

The German road system is classified on the basis of road network functions and traffic quality (capacity) requirements into different groups and categories, as shown in Tables 1 and 2. The following procedure for achieving a consistent road characteristic as a consequence of curvilinear alinement design is valid, first, for two-lane rural roads of Group A and, partially, for those of Group B. For multilane roads and two-lane roads with collector or even local functions, other assumptions become relevant.

The following provides for the first time in a TRB publication a brief overview of the important steps of the German design procedure for two-lane rural roads of Group A and partially for roads of Group B. The reason for this is to compare the German assumptions with the more sophisticated procedure for modern highway design suggested by the authors in the next section.

1. Fundamentally, the design speed V_d shall remain constant for longer roadway sections. Thus, the road characteristic should be well balanced for a driver along the course of the road section. This is a basic assumption for achieving curvilinear alinement. If, along the course of a longer road section, a change in the road characteristic and a corresponding change in the design speed are necessary, for example, by definite changes in the topography, then in the transition section the design elements should be carefully tuned to each other so that they change only gradually. (Nowhere in the German guidelines is the term *longer road section* defined.)

2. Besides the design speed, the operating speed, expressed by the 85th percentile speed (V_{85}), should also be consistent along the selected road section. First, this is achieved by using the required sequences of curves shown in Figure 3. This figure is based solely on experience. It defines the ranges in which the radii of two successive circular curves in either the same or the opposite direction for roads of Group A and Category BII would need to fall to reach a well-balanced relationship for safety reasons; this is also desirable for roads of Category BIII and, if possible, Category BIV (Table 2).

As an example for Figure 3, when a curve with a radius of 500 m (1,640 ft) is combined with curves with the following radii (*R*), one obtains the indicated range classifications:

R = 200 m (600 ft) falls into the avoidable range,

R = 300 m (1,000 ft) falls into the fair range,

R = 350 m (1,200 ft) falls into the good range, and

R = 400 m (1,300 ft) falls into the very good range.

For roads of Categories AI and AII (major connector function), the sequences of the radii or curves fall into the very good or good range. For roads of Categories AIII, AIV, and BII (minor connector function), the fair range is sufficient. A sequence of radii falling into the fair range is also desirable for roads of Categories BIII and BIV.

Such a tuning of radii of curve sequences is called *relation design*, the design method to strive for today. Relation design means that no more single design elements with minimum or maximum limiting values are put together more or less arbitrarily. Design element sequences will be formed, and the design elements following one another in these sequences must be subject to certain relations corresponding to Figure 3. In this way the evaluation of alinements becomes possible and comparisons between design speed (Table 2, column 7) and operating speeds (V_{85}) can be made according to Figure 4.

All curves in Figure 3 are extrapolated down to a curve with a radius of 50 m (164 ft), since this value may still exist on some state and federal roads in Germany.

Besides these assessments for circular curve sequences according to Figure 3, for the sequence tangent-transition curve-circular curve, the following minimum radii shall be applied, depending on the length L (in meters) of the tangent, if the design speed V_d does not require a curve with a larger radius (*7*).

Road Category	*Length (m) of Tangent*	*Minimum* R*(m) of Circular Curve*
AI, AII	≥600	>600
	<600	>L
AIII, AIV, BII (BIII, BIV)	≥500	>500
	<500	>L

In addition, the minimum length of a circular curve should be so long that driving through the curve at the design speed will require at least 2 sec.

Finally, the German guidelines suggest that the continuance of the same road group and category over longer road sections is very important for a consistent curvilinear alinement. This is especially true for two-lane rural roads of Group A and Category BII, and sometimes even for roads of Category BIII (Table 2).

TABLE 1 Classification of Roads by Groups

LOCATION	CONCENTRATION OF BUILDINGS	IMPORTANT FUNCTION	GROUP
Rural Areas	Low (or Zero)	Connector	A
Urban Areas	Low (or Zero)	Connector	B
	High	Connector	C
		Collector	D
		Local	E

TABLE 2 **Classification of Roads by Groups and Categories**

ROAD FUNCTION		DESIGN AND OPERATIONAL CHARACTERISTICS				
Category Group	Road Category	Kind of Traffic	Permissible Speed Limit (km/hr)	Cross Section	Intersection Access	Design Speed V_e(km/hr)
1	2	3	4	5	6	7
A: - Low Concentration of Buildings - Rural Areas - Important Connector Functions	AI Statewide or Interstate Functions	Vehicles Vehicles	None ≤100 (120)	Multiple Lane 2 Lane	Controlled (Controlled) Free	120 100 100 90 (80)
	AII Regional Functions	Vehicles All*	None (100) ≤100	Multiple Lane 2 Lane	Controlled (Free) Free	100 90 (80) 90 80 (70)
	AIII Functions Between Municipalities	Vehicles All	≤100 ≤100	Multiple Lane 2 Lane	(Controlled) Free Free	(90) 80 70 80 70 60
	AIV Large Area Accessibility Functions	All	≤100	2 Lane	Free	70 60 (50)
	AV Subordinate Functions	All	≤100	2 Lane	Free	(50) None
B: - Low Concentration of Buildings - Urban or Suburban Areas - Important Connector Functions	BII Primary Arterial	Vehicles	≤80	Multiple Lane	Controlled (Free)	80 70 (60)
	BIII Secondary Arterial	All All	≤70 ≤70	Multiple Lane 2 Lane	Free	70 60 (50) 70 60 (50)
	BIV Main Collector	All	≤60	2 Lane	Free	60 50

(continued on next page)

TABLE 2 *(continued)*

C: - High Concentration of Buildings - Urban Areas - Important Connector Functions	CIII Secondary Arterial	All All	50 (≤70) 50 (≤60)	Multiple Lane 2 Lane	Free	(70) (60) 50 (40) (60) 50 (40)
	CIV Main Collector	All	≤50	2 Lane	Free	50 (40)
D: - High Concentration of Buildings - Urban Areas - Important Collector Functions	DIV Collector	All	≤50	2 Lane	Free	None
	DV Local	All	≤50	2 Lane	Free	None
E: - High Concentration of Buildings - Urban Areas - Important Local Functions	EV Local	All	≤30 Walking Speed	2 Lane	Free	None
	EVI Dwelling Functions	All	Walking Speed	2 Lane	Free	None

*Indicates All Types of Road User Groups Combined (...) *exception

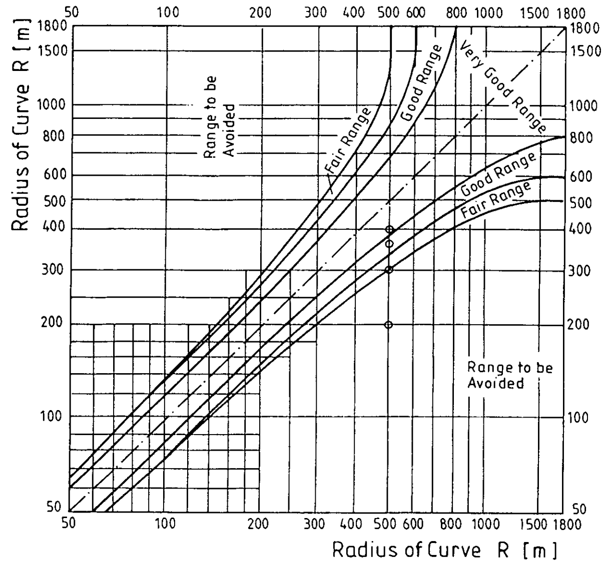

FIGURE 3 Tuning of radii of curve sequences for roads of Group A and Category BII for achieving curvilinear alinements in Germany (*7*).

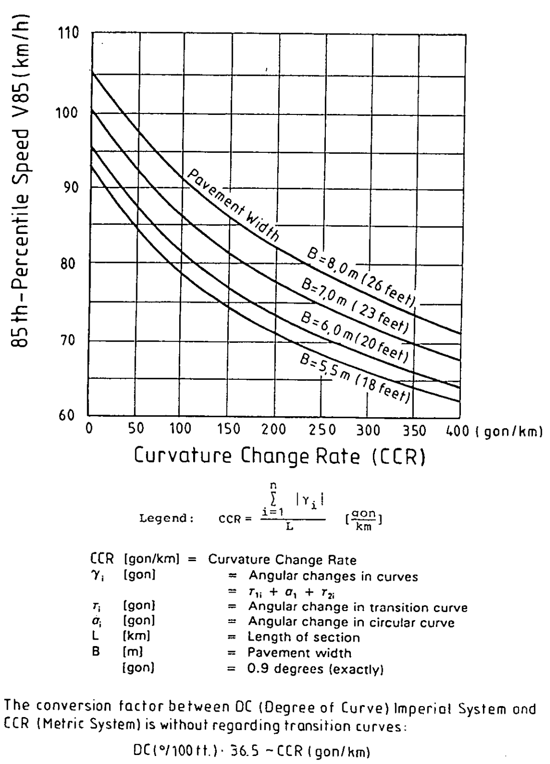

FIGURE 4 Nomogram for evaluating operating speeds (V_{85}) as related to curvature change rate (CCR) for individual pavement widths according to German design guidelines (*7*).

3. The V_{85} can be determined from Figure 4 with regard to the German design parameters curvature change rate (CCR), which is comparable to the U.S. design parameter *degree of curve* (see conversion factor in Figure 4) and lane width. For an observed design element (curve or tangent), the difference between V_{85} and design speed should not exceed

$$V_{85} - V_d \leq 20 \text{ km/hr.}$$

According to Safety Criterion II in Figure 1, this requirement would include good and fair design practices together; that means that a separate evaluation does not exist.

4. However, between two successive road sections (not design elements), the values for V_{85} that are determined should not differ by more than 10 km/hr (6 mph). This requirement would correspond to good design practices according to Safety Criterion I of Figure 1, but for the term *road section,* it is only stated that it should exhibit a similar road characteristic, a statement that is difficult to understand and a requirement that is difficult to achieve.

The discussions of Issues 3 and 4 above reveal that the assessments in Figure 1 appear to be more sophisticated than the German ones. Note also that the ranges for different design practices in Figure 3 are based on experience only and not on analytical derivations.

A detailed example application regarding Issues 1 to 4 discussed above is presented in a paper by Lammet al. in this Record.

The safety of traffic flow depends on numerous, partially unassessable influences. Besides traffic volume and composition, the design of the cross section of the road is also of significance.

In the German *Guidelines for the Design of Roads, Part: Cross Sections (RAS-Q)* (*18*), road characteristics and safety are discussed in the following way. Road cross section, alinement, and intersections are essential parts of the road characteristic and together influence traffic safety. Therefore, they must be tuned to each other. A cross section inconsistency may be the result of upgrading a highway cross section without upgrading the alinement. Because cross-section features can be more apparent than the alinement, there may be instances in which a wider cross section on an old alinement might convey a message to the driver that could lead to an inappropriate expectancy on the basis of visual aspects of the cross section. Therefore, cross-section features are very important for the road characteristic.

Furthermore, for a sound curvilinear alinement, the designer must be concerned not only with the horizontal alinement but also with the vertical alinement as well as their superimposition or coordination. The essentials of alinement coordination are reported by Smith and Lamm in a paper in this Record.

SUGGESTED PROCEDURE FOR MODERN HIGHWAY DESIGN

For achieving a good curvilinear alinement, Safety Criteria I and II (Figure 1) are of significant importance. They are based on research conducted in the United States and Germany to determine whether these criteria could be adopted for modern practice in new design, redesign, major reconstruction, and resurfacing, restoration, or rehabilitation (RRR) projects on both continents (*12*).

Research evaluated the impact of design parameters, degree of curve, curvature change rate, length of curve, superelevation rate, lane width, shoulder width, sight distance, gradient up to 6 percent, and traffic volume on a data base of 322 two-lane, curved highway sections in New York State (*8–11*) and one data base of 204 sections in Germany (*19,20*). The present data bases contain roadway sections with gradients of up to 6 percent and traffic volumes of between 500 and 10,000 vehicles per day. The research demonstrated that the most successful parameter in explaining much of the variability in operating speeds (V_{85}) and accident rates was degree of curve and curvature change rate. The relationship between operating speed and degree of curve was quantified by regression models and is schematically shown in Figure 5 for the United States.

With respect to degree of curve, V_{85} can be determined for every curve or independent tangent by using Figure 4. An *independent tangent* [for defining and classifying independent tangents (*21*)] is classified to be long enough to be regarded in the curve-tangent-curve design process as an independent design element, whereas a short tangent is called *nonindependent* and can be neglected. By knowing the V_{85} of every element, the absolute speed differences between successive design elements can be calculated. The observed road section, consisting of sequences tangent to curve or curve to curve, can then be classified as being of good, fair, or poor design (Figure 1). Operating speed backgrounds (like those in Figure 4 or 5) should be part of every modern geometric highway design guideline when striving for a good curvilinear alinement, as will be explained in the following section.

Safety Criterion I

For achieving sound transitions between successive design elements, the recommended ranges for good, fair, and poor design practices are given in Figure 1 on the basis of the absolute differences in the corresponding V_{85}s. They provide a quantifiable and sophisticated classification system and are largely based on mean accident rates (*10,11*).

- Good design practice means that, according to the ranges in Figure 1, consistency in horizontal alinement exists between successive design elements for these road sections and that the horizontal alinement does not create inconsistencies in vehicle operating speed. A curvilinear alinement can be expected.
- Fair design practice means that these road sections may contain at least minor inconsistencies in geometric design between successive design elements. Normally, they would warrant traffic warning devices but not redesigns.
- Poor design practice means that these road sections have strong inconsistencies in horizontal geometric design between successive design elements combined with those breaks in the speed profile that may lead to critical driving maneuvers. A noncurvilinear alinement must be expected. Normally, redesigns are recommended.

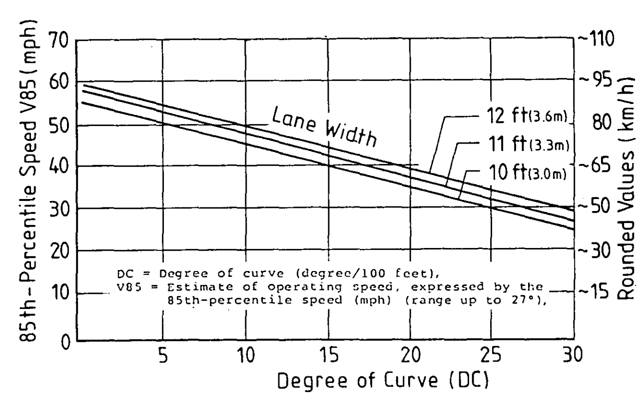

FIGURE 5 Nomogram for evaluating operating speeds (V85) as related to degree of curve for individual lane widths according to U.S. standards (*10,11*).

Safety Criterion II

For evaluating single design elements like curves and independent tangents, the recommended ranges for good, fair, and poor design practices are given in Figure 1 on the basis of the absolute difference between the observed V_{85} and the design speed.

V_{85} can be determined for the observed curved roadway section by using Figure 4 (Germany) or Figure 5 (United States). This time, however, the V_{85} of the circular curve itself or the independent tangent is of prime importance for new designs or examining old designs, for example, in cases of major reconstruction or RRR projects.

In Germany V_d is determined depending on the classification of roads in Table 2. In the United States the following design speeds (where 1 mph = 1.61 km/hr) are recommended in the Green Book (*3*):

Functional Type of Road	*Design Speed (mph)*
Local rural roads	30–50
Rural collectors	40–60
Rural arterial	40–70
Urban arterial	40–60
Urban freeways	50–60
Rural freeways	70

The variation in design speeds for a given road type generally depends on the type of terrain, driver expectancy, and in some cases, design hour volumes.

- Good design practice means that, according to the ranges in Figure 1, no adaptations or corrections between V_{85} and design speed are necessary. A curvilinear alinement can be expected.
- Fair design practice means that, for example, in the case of RRR projects, superelevation rates should be related to the V_{85} and not to the design speed to ensure that the assumed side friction will accommodate side friction demand. In cases of resurfacing projects, high skid resistance values should be required.
- Poor design practice means that redesigns are usually recommended. A noncurvilinear alinement must be expected.

TUNING OF RADII OF CURVE SEQUENCES

On the basis of the recommended changes in operating speeds (V_{85}) for the different design levels of Criterion I in Figure 1, the relationships in Figure 6 were developed. Contrary to the German relationships in Figure 3, which were gained more or less by experience, the boundaries for good, fair, and poor design in Figure 6 were precisely calculated for the assumed operating speed dif-

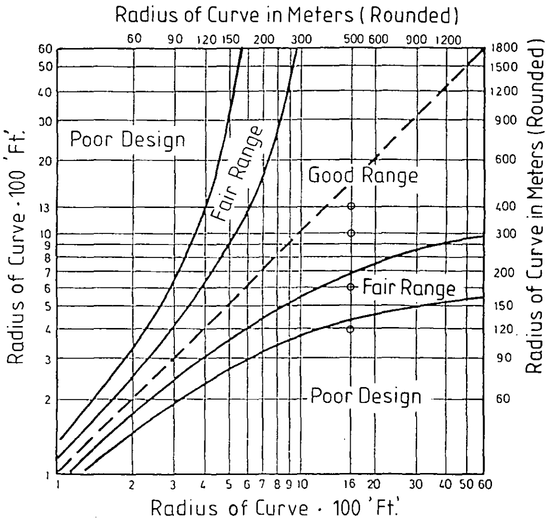

FIGURE 6 Tuning of radii of curve sequences for good and fair design as well as for detecting poor design practices on the basis of the U.S. operating speed background.

ferences in Figure 1 and the corresponding changes in degree of curve, as explained previously (*8*). The degree of curve values was then converted to radii of curve. For instance, Figure 6 shows the tuning of radii of curve sequences in the same or in the opposite direction for possible designs.

As an example for Figure 6, when a curve with a radius of about 500 m (1,600 ft) (see example for Figure 3) is combined with curves with the following radii (*R*), one obtains the indicated design classifications:

R = 120 m (400 ft) falls into the poor design,

R = 180 m (600 ft) falls into the fair design,

R = 300 m (1,000 ft) falls into the good design, and

R = 400 m (1,300 ft) also falls into the good design.

Regarding a tangent-to-curve sequence, the boundaries of good designs correspond to curves with calculated radii of about 350 m ($\geq$1,200 ft). Thus curves with radii of $\geq$350 m ($\geq$1,200 ft) should follow an independent tangent (*21*) not to create inconsistencies in vehicular operating speeds. The boundaries of fair designs correspond to curves with calculated radii of 350 m (1,200 ft) > R > about 200 m (600 ft). Radii within this range should follow an independent tangent in the tangent-to-curve sequence for fair design practices. These values agree well with the minimum radii for design speeds of 97 km/hr (60 mph) (good design) and of about 72 km/hr (45 mph) (fair design) for a superelevation rate of 8 percent in Table III-6 of the Green Book (*3*).

Thus, in comparison with the example application of Figure 3, the example application of Figure 6 allows the use of a combination of a wider range of curve sequences for fair and good design practices. This may be because the relationships in Figure 6 correspond exactly to the assumptions of Criterion I in Figure 1 on the basis of the U.S. operating speed background in Figure 5. The German relationships are based on experience only. Furthermore the operating speeds (V_{85}) in Germany are higher than those in the United States. This leads automatically to higher V_{85} differences between successive design elements and results in a smaller range of permissible curve sequences for fair and good design practices in Germany (compare Figures 3 and 6).

These statements should clarify that operating speed backgrounds like those used for Figures 4 and 5 depend significantly on the driving behaviors of motorists in the specific country under study.

Consequently, the diagrams for tuning of radii of curve sequences for different design levels should be developed on the basis of operating speed backgrounds, as was demonstrated for the United States in this paper and previously (*8*). Therefore, those evaluation backgrounds for achieving a sound relation design like those in Figures 3 and 4 for Germany and Figures 5 and 6 for the United States are important assumptions for every country when establishing modern geometric design guidelines for highways.

For example, by applying Figure 6, the U.S. highway designer could immediately decide whether certain radii of succeeding curves fall into the range of good, fair, or poor design practices.

Thus, for achieving gentle curvilinear alinements in cases of new designs, major reconstruction, and RRR projects, the highway engineer should examine horizontal alinement by

- Safety Criterion I according to Figure 1,
- Safety Criterion II according to Figure 1, and
- The design ranges according to Figure 6.

If all three evaluation procedures fall into the good design range, it can be said definitely that a good and sound curvilinear alinement exists. Normally, the results of Safety Criterion I and for the design ranges in Figure 6 correspond to each other, since both evaluation procedures depend on similar assumptions. The results of Safety Criterion II, however, must be regarded as fully independent.

In the same way, existing two-lane rural roads can also be classified for detecting fair and poor design practices to evaluate endangered (fair) and dangerous (poor) road sections.

APPLICATIONS FOR THE GREEN BOOK

Curvilinear alinement design as described here will allow the designer to quantitatively determine if an alinement is consistent or the alinement change necessary for the required consistency meets driver expectancy to achieve safer operation. This quantification of consistency in terms of Safety Criteria I and II in Figure 1 as well as the proposed design ranges according to Figure 6 will allow the designer to evaluate the effects of fitting together geometric components conveniently and economically and to satisfy operational requirements. Curvilinear alinement design is applicable to new designs, the evaluation of existing designs, and RRR projects.

In new designs the curvilinear alinement design process should be of specific assistance in quantifying the effects of the following of the nine General Controls for Horizontal Alinement listed in the Green Book (*3*):

2. In alinement predicated on a given design speed, use of maximum curvature for that speed should be avoided wherever possible. The designer should attempt to use generally flatter curves, retaining the maximum for the most critical conditions.

3. Consistent alinement should always be sought. . . . Curvilinear Alinement design will enable the designer to quantify the consistency of alinements to effectively transition the alinement for "the most critical conditions" in a consistent fashion and to quantitatively determine how sharp a curve to satisfactorily use at the end of a long tangent.

6. Caution should be exercised in the use of compound curves, . . . compound curves with large differences in curvature introduce the same problems that arise at a tangent approach to a circular curve. . . .

7. An abrupt reversal in alinement should be avoided. . . . Curvilinear alinement design can assist in quantifying the level of consistency of various alinement designs in 6 and 7.

CONCLUSION

This paper presented curvilinear alinement design (relation design) as a useful, workable tool for achieving a more consistent road design and thereby avoiding potential safety errors than is generally attained by the individual element, maximum-minimum design approach. This is true for new alinements, upgraded highway alinements, or full-blown RRR projects of two-lane rural roads.

The proposed curvilinear alinement design process is based on quantifiable and sophisticated criteria for evaluating operating speed changes between successive design elements and for tuning operating speeds and design speeds of single design elements to each other.

It is recommended that curvilinear alinement design be evaluated for inclusion as a recommended design process in the Green Book (*3*).

The practical fundamentals described in this paper were extended in another paper by Lamm et al. in this Record. Both papers should be regarded as one unit.

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Horizontal Curve Design for Passenger Cars and Trucks

DOUGLAS W. HARWOOD AND JOHN M. MASON, JR.

The adequacy of the 1990 AASHTO geometric design policy for safely accommodating both passenger cars and trucks on horizontal curves is evaluated. The evaluation includes both the high-speed or open-highway horizontal curve design criteria in AASHTO Green Book Table III-6 and the low-speed design criteria for intersections and turning roadways in AASHTO Green Book Table III-17. The evaluation of current horizontal curve design policy is conducted by means of a sensitivity analysis that evaluates the margin of safety against vehicle skidding and rollover for both passenger cars and trucks traveling at the design speed on minimum-radius curves designed in accordance with AASHTO policy. It is concluded that current AASHTO horizontal curve design policy for rural highways and high-speed urban streets in Green Book Table III-6 provides an adequate margin of safety against both skidding and rollover as long as vehicles do not exceed the design speed of the curve. However under nearly worst-case conditions, skidding and rollover can occur on a horizontal curve, particularly at lower design speeds, if vehicles exceed the design speed by only a small amount. This finding suggests the importance of selecting a realistic horizontal curve design speed that will not be exceeded by substantial portions of the traffic stream. The AASHTO horizontal curve design criteria for intersections and turning roadways and low-speed urban streets, presented in Green Book Tables III-16 and III-17, are generally adequate for passenger cars that do not exceed the design speed, but may not be adequate for all trucks.

Geometric design policy for horizontal curves is presented in the AASHTO Green Book (*1*). This paper evaluates the adequacy of these existing criteria for safely accommodating both passenger cars and trucks on horizontal curves. The paper presents an analysis of the effect of minimum radius of curvature and maximum superelevation rate on the margins of safety against vehicle skidding and rollover and the vehicle speeds at which skidding and rollover will occur.

This paper focuses on the adequacy for passenger cars and trucks of the high-speed or open-highway design criteria for horizontal curves presented in AASHTO Green Book Table III-6. The evaluation of these high-speed design criteria is based on an analysis by Harwood et al. (*2*) from a recent FHWA study. The paper also assesses the adequacy for passenger cars and trucks of the low-speed horizontal curve design criteria that apply to urban streets, intersections, and turning roadways and that are presented in Green Book Tables III-16 and III-17.

CURRENT HIGH-SPEED DESIGN CRITERIA FOR HORIZONTAL CURVES

Under the current AASHTO policy, a vehicle on a horizontal curve is represented as a point mass. From the basic laws of Newtonian physics, the lateral acceleration of a point mass traveling at constant speed on a circular path can be represented by the relationship:

$$a = \frac{V^2}{15R} \tag{1}$$

where

a = lateral acceleration (g),
V = vehicle speed (km/hr or mph), and
R = radius of curve (m or ft)

The lateral acceleration is expressed in units of the acceleration of gravity (g), which is equal to 9.8 m/s^2 (32.2 ft/s^2). On a superelevated curve, the superelevation offsets a portion of the lateral acceleration, such that:

$$a_{\text{net}} = \frac{V^2}{15R} - e \tag{2}$$

where a_{net} is unbalanced portion of lateral acceleration (g) and e is superelevation (ft/ft). The unbalanced portion of the lateral acceleration of the vehicle is a measure of the forces acting on the vehicle that tend to make it skid off the road or overturn. The side frictional demand of the vehicle is mathematically equivalent to the unbalanced lateral acceleration (a_{net}). For this reason Equation 2 appears in the Green Book in the form:

$$f = \frac{V^2}{15R} - e \tag{3}$$

where f is side friction demand. The tendency of the vehicle to skid must be resisted by tire-pavement friction. The vehicle will skid off the road unless the tire-pavement friction coefficient exceeds the side friction demand. However it is also critical for safe vehicle operations that vehicles not roll over on horizontal curves. The tendency of the vehicle to overturn must be resisted by the roll stability of the vehicle. The vehicle will roll over unless the rollover threshold of the vehicle exceeds the unbalanced lateral acceleration (a_{net}).

Selection of Radius and Superelevation

The objective of AASHTO criteria for horizontal curve design is to select the radius and superelevation so that the unbalanced lateral acceleration is kept within comfortable limits. AASHTO policy limits the unbalanced lateral acceleration for horizontal curves

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to a maximum of 0.17 g at 32 km/hr (20 mph), decreasing to a maximum of 0.10 g at 113 km/hr (70 mph). This limitation is based on the results of research performed in 1936 through 1949 that established 0.17 g as the maximum unbalanced lateral acceleration at which drivers felt comfortable (see Green Book Figure III-5). Thus it is important to note that these AASHTO criteria are based on maintaining comfort levels for passenger car drivers and passengers.

The AASHTO Green Book provides design charts for maximum superelevation rates (e_{max}) from 0.04 to 0.12. Highway agencies have established their own policies concerning the maximum superelevation rate that will be used on horizontal curves. Most highway agencies use maximum superelevation rates of either 0.06 or 0.08; states that experience snow and ice conditions typically use lower superelevation rates. For any particular maximum superelevation rate and maximum side friction demand, the minimum radius of curvature can be determined as:

$$R_{min} = \frac{V_d^2}{15\,(e_{max} + f_{max})} \tag{4}$$

where

R_{min} = minimum radius of curvature (m or ft),
V_d = design speed of curve (km/hr or mph),
e_{max} = specified maximum superelevation rate (m/m or ft/ft), and
f_{max} = specified maximum side friction demand.

Table 1 presents the minimum radius of curvature for specific combinations of maximum superelevation rate and maximum side friction demand recommended by AASHTO. The maximum side friction demand (f_{max}) values that appear in Table 1 are the maximum comfortable lateral acceleration values recommended by AASHTO for high-speed rural highways and urban streets. The data in the tables in this paper are presented in customary units for consistency with the 1990 Green Book.

In the design of a horizontal curve under AASHTO policy, the first major decision is to select its radius of curvature. Next the selected radius is checked to ensure that it is not less than R_{min} for the design speed of the highway. Finally if the selected radius is greater than R_{min}, a superelevation less than e_{max} is selected by using Tables III-8 through III-12 of the AASHTO Green Book.

TABLE 1 AASHTO Criteria for Maximum Degree of Curve and Minimum Radius for Horizontal Curves on Rural Highways and High-Speed Urban Streets (*1*)

Design Speed (mph)	Maximum e	Maximum f	Total (e+f)	Maximum Degree of Curve	Rounded Maximum Degree of Curve	Maximum[a] Radius (ft)
20	.04	.17	.21	44.97	45.0	127
30	.04	.16	.20	19.04	19.0	302
40	.04	.15	.19	10.17	10.0	573
50	.04	.14	.18	6.17	6.0	955
55	.04	.13	.17	4.83	4.75	1,186
60	.04	.12	.16	3.81	3.75	1,528
20	.06	.17	.23	49.25	49.25	116
30	.06	.16	.22	20.94	21.0	273
40	.06	.15	.21	11.24	11.25	509
50	.06	.14	.20	6.85	6.75	849
55	.06	.13	.19	5.40	5.5	1,061
60	.06	.12	.18	4.28	4.25	1,348
65	.06	.11	.17	3.45	3.5	1,637
70	.06	.10	.16	2.80	2.75	2,083
20	.08	.17	.25	53.54	53.5	107
30	.08	.16	.24	22.84	22.75	252
40	.08	.15	.23	12.31	12.25	468
50	.08	.14	.22	7.54	7.5	764
55	.08	.13	.21	5.97	6.0	960
60	.08	.12	.20	4.76	4.75	1,206
65	.08	.11	.19	3.85	3.75	1,528
70	.08	.10	.18	3.15	3.0	1,910
20	.10	.17	.27	57.82	58.0	99
30	.10	.16	.26	24.75	24.75	231
40	.10	.15	.25	13.38	13.25	432
50	.10	.14	.24	8.22	8.25	694
55	.10	.13	.23	6.53	6.5	877
60	.10	.12	.22	5.23	5.25	1,091
65	.10	.11	.21	4.26	4.25	1,348
70	.10	.10	.20	3.50	3.5	1,637
20	.12	.17	.29	62.10	62.0	92
30	.12	.16	.28	26.65	26.75	214
40	.12	.15	.27	14.46	14.5	395
50	.12	.14	.26	8.91	9.0	637
55	.12	.13	.25	7.10	7.0	807
60	.12	.12	.24	5.71	5.75	996
65	.12	.11	.23	4.66	4.75	1206
70	.12	.10	.22	3.85	3.75	1528

NOTE: In recognition of safety considerations, use of e_{max} = 0.04 should be limited to urban conditions.
[a]Calculated using rounded maximum degree of curve.

Figure III-9 of the AASHTO Green Book summarizes the superelevation rates used for curves with radii greater than R_{min}.

Transition Design

Most horizontal curves are circular curves that directly adjoin tangent roadway sections at either end with no transition curve. Thus a vehicle entering a curve theoretically encounters an instantaneous increase in lateral acceleration from a minimal level of the tangent section to the full lateral acceleration required to track the particular curve. The opposite occurs as a vehicle leaves a horizontal curve. In fact there is a gradual (rather than an instantaneous) change in lateral acceleration, because drivers steer a spiral or transition path as they enter or leave a horizontal curve. The design of the superelevation transition section is used to partially offset the changes in lateral acceleration that do occur. First, a superelevation runout section is used on the tangent roadway to remove the adverse crown slope. Next a superelevation runoff section is provided in which the pavement is rotated around its centerline or inside edge to attain the full required superelevation; typical design practice is to place two-thirds of the superelevation runoff on the tangent approach and one-third on the curve. Table III-15 in the AASHTO Green Book presents the required length for superelevation runoff on two-lane pavements.

The AASHTO Green Book encourages the use of spiral transition curves to provide a smooth transition between tangents and circular curves. In a spiral curve, the degree of curvature varies linearly from zero at the tangent end to the degree of the circular arc at the circular curve end. The length of the spiral curve can be made the same as that of the superelevation runoff, so that the degree of curvature and pavement cross-slope change together.

DISCUSSION OF HORIZONTAL CURVE DESIGN POLICY

Consideration of Driver Comfort

The authors are concerned that nearly 50 years have passed since the completion of the research on driver comfort levels on which the AASHTO policy is based. Obviously, vehicle design has changed dramatically since 1949. There is a definite need for research to reevaluate the driver comfort levels used in AASHTO policy, and the authors understand that FHWA plans to conduct such research in the near future. This research may identify a need for changes in the AASHTO horizontal curve design policy to ensure driver comfort.

Beyond the concern for maintaining comfortable levels of lateral acceleration for drivers, there is a safety concern in minimizing accidents associated with vehicle skidding or rollover. The AASHTO horizontal curve design criteria are not based explicitly on estimates of available tire-pavement friction levels or vehicle rollover thresholds. Rather it is assumed implicitly that the available friction levels and rollover thresholds are higher than the specified driver comfort levels. A driver who chooses to exceed the design speed of a curve will experience a level of lateral acceleration that may make him or her uncomfortable. This lack of comfort does not, by itself, necessarily create a safety problem, since an accident need not occur just because a driver chooses to experience a slightly uncomfortable level of lateral acceleration. Furthermore drivers who consistently choose to exceed the design speeds of horizontal curves are probably those with a higher tolerance for lateral acceleration. Safety concerns enter if the driver chooses a travel speed on a horizontal curve that could lead to vehicle skidding or rollover. The review of horizontal curve design policy in the remainder of this paper examines the adequacy of the margins of safety against rollover for vehicles traversing minimum-radius horizontal curves at speeds at or above the design speed.

Consideration of Friction Demand

The point mass representation of a vehicle that forms the basis for Equations 1 to 3 is not based on any particular set of vehicle characteristics and is theoretically as applicable to trucks as it is to passenger cars. However in light of the differences between passenger cars and trucks in size, number of tires, tire characteristics, and suspension characteristics, the suitability for trucks of the point mass assumption was recently reexamined.

A 1985 FHWA study (*3*) found that the point mass representation in Equation 3 can be used to determine the net side friction demand of both passenger cars and trucks, because the basic laws of physics apply to both. However that study found that although the friction demands at the four tires of a passenger car are approximately equal, the friction demands at the various tires of a tractor-trailer truck vary widely. The net result of this tire-to-tire variation in friction demand is that trucks typically demand approximately 10 percent higher side friction than passenger cars. The authors have termed this higher side friction demand the *effective side friction demand* of trucks.

The point mass representation of a vehicle has another weakness, however, that applies to both passenger cars and trucks. Equation 3 is based on the assumption that vehicles traverse curves following a path of constant radius equal to the radius of the curve. However field studies have shown that all vehicles oversteer at some point on a horizontal curve. At the point of oversteering, the vehicle is following a path radius that is less than the radius of the curve (*4*). Thus at some point on each curve, the friction demand of each vehicle will be slightly higher than that suggested by Equation 3. Oversteering by passenger cars is not considered in the AASHTO design policy for horizontal curves, but it is probably not critical because the AASHTO maximum lateral acceleration requirements are based on driver comfort levels rather than the available pavement friction. No data are available on the amount of oversteering by trucks relative to passenger cars.

Consideration of Rollover Threshold

The AASHTO criteria for horizontal curve design do not explicitly consider vehicle rollover thresholds. The rollover threshold for passenger cars may be as high as 1.2 *g*, so a passenger car will normally skid off a road long before it would roll over (*5*). Thus the consideration of rollover threshold is not critical for passenger cars. However, tractor-trailer trucks have relatively high centers of gravity and consequently tend to have low rollover thresholds. Furthermore because of suspension characteristics, the rollover threshold of tractor-trailer trucks is substantially less than it would be if a truck were a rigid body.

Recent research has determined the rollover thresholds of a number of common trucks with typical loading configurations (*2,6,7*). Some trucks with rollover thresholds of as low as 0.30 *g* are found on the road. Since AASHTO design policy permits a lateral acceleration of as great as 0.17 *g*, the margin of safety for trucks with low rollover thresholds on some horizontal curves is not great. Furthermore as discussed above, oversteering will generally result in a lateral acceleration greater than f_{max} at some point on the curve for vehicles traveling at the design speed, which will tend to reduce the margin of safety.

EVALUATION OF MARGIN OF SAFETY FOR PASSENGER CARS AND TRUCKS

An evaluation has been conducted to determine whether the existing AASHTO high-speed horizontal curve design criteria are adequate to keep both passenger cars and trucks from skidding off the road and to keep them from rolling over.

Margins of Safety Against Skidding

Current design criteria for horizontal curves are intended to keep vehicles from skidding off the road on wet pavements. The criteria are based on the standard curve formula in Equation 3, which provides that a portion of the lateral acceleration developed by the vehicle will be resisted by superelevation and the remainder will be resisted by tire-pavement friction.

The margin of safety against skidding for a passenger car or truck on a horizontal curve is defined as the difference between the available tire-pavement friction and the friction demand of the vehicle as it tracks the curve. Friction demand is the portion of the vehicle's lateral acceleration that is not offset by superelevation. The margin of safety represents the additional lateral acceleration that the vehicle could undergo without skidding. The objective of the analysis is to determine the margin of safety against skidding for both passenger cars and trucks on minimum-radius curves designed in accordance with AASHTO criteria. Horizontal curves with longer radii would have larger margins of safety than those calculated here.

The assumptions made in computing the margin of safety against skidding for passenger cars and trucks are as follows:

- Both passenger cars and trucks traverse the curve at the design speed on a path that follows a constant radius equal to the radius of the curve.
- The pavement has a relatively poor wet pavement friction level equivalent to the pavement assumed by AASHTO for stopping sight distance (see locked-wheel braking coefficients for passenger cars for specific design speeds in Green Book Table III-1). These range from a braking friction coefficient of 0.40 at 32 km/hr (20 mph) to 0.28 at 113 km/hr (70 mph). The locked-wheel braking coefficient for a dry pavement is assumed to be 0.65. The cornering friction coefficient at a specific speed is assumed to be 1.45 times the locked-wheel braking coefficient (*1,8*).
- The tire pavement friction generated by truck tires is only 70 percent of that generated by passenger car tires (*8*).
- The effective friction demand for a truck is 10 percent higher than that for a passenger car, as discussed above (*2,3*).

A simple example will show how the margin of safety against skidding is calculated. A horizontal curve with a design speed of 32 km/hr (20 mph) and a maximum superelevation rate of 0.04 has a maximum tolerable lateral acceleration (f_{max}) of 0.17 *g*, in accordance with Green Book Table III-6. The minimum radius derived from Equation 4 is

$$R_{min} = \frac{20^2}{15\,(0.04 + 0.17)} = 127 \text{ ft } (39 \text{ m})$$

The friction demand for a passenger car traversing this curve at the design speed is equivalent to f_{max} (0.17).

The available tire-pavement friction under wet pavement conditions is 1.45 times the assumed AASHTO locked-wheel braking coefficient of 0.40: (0.40) (1.45) = 0.58.

The margin of safety for a passenger car is 0.58 − 0.17 = 0.41. In other words, a passenger car could undergo 0.41 *g* of additional lateral acceleration without skidding.

For a truck, the effective friction demand would be 10 percent higher than that for a passenger car: (0.17) (0.10) = 0.19.

The tire-pavement friction for a truck is only 70 percent of that for a passenger car: (0.58) (0.70) = 0.41. Therefore, the margin of safety for a truck is 0.41 − 0.19 = 0.22. In other words, a truck could undergo additional lateral acceleration of only 0.22 *g* without skidding, in contrast to 0.41 *g* for a passenger car.

The results of similar calculations for design speeds from 32 to 113 km/hr (20 to 70 mph) and a maximum superelevation rates from 0.04 to 0.10 are presented in Table 2. Table 2 shows that the current AASHTO criteria provide a margin of safety of 0.31 to 0.41 *g* against a passenger car skidding when traveling on wet pavement at the design speed of a minimum-radius curve. Table 2 also shows that the margins of safety against skidding on dry pavement are much higher than those on wet pavement.

The margins of safety against skidding by trucks are in the range of from 0.17 to 0.22 *g*, which is lower than that for passenger cars. A later section of the paper expresses these results in terms of the vehicle speed at which skidding would occur. The margins of safety for passenger cars and trucks in Table 2 are large enough to provide safe operations if there are no major deviations from the basic assumptions used in horizontal curve design. The effects of such deviations are considered below.

Margin of Safety Against Rollover

The margin of safety against rollover is the magnitude of the additional lateral acceleration that a vehicle could undergo without rolling over. The rollover margin of safety has been computed on the basis of the following assumptions:

- Passenger cars have very high rollover thresholds, possibly as high as 1.2 *g* (*5*).
- The most unstable trucks have rollover thresholds in the range of from 0.27 to 0.40 *g*. Most trucks have substantially higher rollover thresholds.

The following example shows how the margin of safety against rollover is calculated by using the same computational example as that given above for the minimum-radius curve for a 32-km/hr (20-mph) design speed and a maximum superelevation rate of 0.04. The lateral acceleration for a passenger car traversing such

TABLE 2 Margins of Safety Against Skidding on Horizontal Curves (2)

		Passenger car							Truck					
Design speed (mph)	Maximum super-elevation e	Maximum comfortable lateral acceleration (g)	Maximum demand f	Minimum radius (ft)	Available f (wet)	Margin of safety (wet)	Margin of safety (dry)	Maximum comfortable lateral acceleration (g)	Minimum radius (ft)	Maximum demand f	Truck available f (wet)	Margin of safety (wet)	Margin of safety (dry)
20	0.04	0.17	0.17	127	0.58	0.41	0.77	0.17	127	0.19	0.41	0.22	0.47
30	0.04	0.16	0.16	302	0.51	0.35	0.78	0.16	302	0.18	0.36	0.18	0.48
40	0.04	0.15	0.15	573	0.46	0.31	0.79	0.15	573	0.17	0.32	0.16	0.49
50	0.04	0.14	0.14	955	0.44	0.30	0.80	0.14	955	0.15	0.30	0.15	0.51
60	0.04	0.12	0.12	1,528	0.42	0.30	0.82	0.12	1,528	0.13	0.29	0.16	0.53
20	0.06	0.17	0.17	116	0.58	0.41	0.77	0.17	116	0.19	0.41	0.22	0.47
30	0.06	0.16	0.16	273	0.51	0.35	0.78	0.16	273	0.18	0.36	0.18	0.48
40	0.06	0.15	0.15	509	0.46	0.31	0.79	0.15	509	0.17	0.32	0.16	0.49
50	0.06	0.14	0.14	849	0.44	0.30	0.80	0.14	849	0.15	0.30	0.15	0.51
60	0.06	0.12	0.12	1,348	0.42	0.30	0.82	0.12	1,348	0.13	0.29	0.16	0.53
70	0.06	0.10	0.10	2,083	0.41	0.31	0.84	0.10	2,083	0.11	0.28	0.17	0.55
20	0.08	0.17	0.17	107	0.58	0.41	0.77	0.17	107	0.19	0.41	0.22	0.47
30	0.08	0.16	0.16	252	0.51	0.35	0.78	0.16	252	0.18	0.36	0.18	0.48
40	0.08	0.15	0.15	468	0.46	0.31	0.79	0.15	468	0.17	0.32	0.16	0.49
50	0.08	0.14	0.14	764	0.44	0.30	0.80	0.14	764	0.15	0.30	0.15	0.51
60	0.08	0.12	0.12	1,206	0.42	0.30	0.82	0.12	1,206	0.13	0.29	0.16	0.53
70	0.08	0.10	0.10	1,910	0.41	0.31	0.84	0.10	1,910	0.11	0.28	0.17	0.55
20	0.10	0.17	0.17	99	0.58	0.41	0.77	0.17	99	0.19	0.41	0.22	0.47
30	0.10	0.16	0.16	231	0.51	0.35	0.78	0.16	231	0.18	0.36	0.18	0.48
40	0.10	0.15	0.15	432	0.46	0.31	0.79	0.15	432	0.17	0.32	0.16	0.49
50	0.10	0.14	0.14	694	0.44	0.30	0.80	0.14	594	0.15	0.30	0.15	0.51
60	0.10	0.12	0.12	1,091	0.42	0.30	0.82	0.12	1,910	0.13	0.29	0.16	0.53
70	0.10	0.10	0.10	1,637	0.41	0.31	0.84	0.10	1,637	0.11	0.28	0.17	0.55

a curve at the design speed is f_{max}, or 0.17 g. The margin of safety against rollover by a passenger car on this horizontal curve is calculated as 1.20 − 0.17 = 1.03. A passenger car could undergo an additional lateral acceleration of 1.03 g without rolling over.

The lateral acceleration for a truck traversing the same minimum-radius curve while traveling at the design speed is also equal to 0.17 g. [The 10 percent increase in friction demand for trucks, based on the work by MacAdam et al. (*3*), is applicable only to the consideration of tire-pavement friction in skidding calculations and, thus, does not enter into the rollover calculations.] A truck with a rollover threshold of 0.30 g traversing the curve at the design speed would have a much smaller margin of safety against rollover than a passenger car: 0.30 − 0.17 = 0.13. This truck could undergo an additional lateral acceleration of only 0.13 g without rolling over.

The results of similar calculations for design speeds of from 32 to 113 km/hr (20 to 70 mph) and maximum superelevation rates of from 0.04 to 0.10 are presented in Table 3. Table 3 shows the margins of safety against rollover in units of the acceleration of gravity (g) for passenger cars and for trucks with rollover thresholds (RT) of 0.27, 0.30, 0.35, and 0.40 g.

Table 3 shows that the margin of safety against rollover for passenger cars traveling at the design speed ranges from 1.03 to 1.10 g. At all design speeds, the margin of safety against rollover for a passenger car is much higher than the margin of safety against skidding on either a wet or a dry pavement. Thus, rollover is not a major concern for passenger cars because, unless they collide with another vehicle or object, they will skid rather than roll over. In contrast to the related issue of skidding off the road, the margin of safety against rollover is not dependent on whether the pavement is wet or dry.

A conservative value of the truck rollover threshold appropriate for use in design is 0.30 g. The margin of safety for a truck with a rollover threshold of 0.30 g ranges from 0.13 to 0.20 g. This margin of safety is adequate to prevent rollover for trucks traveling at or below the design speed. The margin of safety against rollover increases with increasing design speed, whereas the margin of safety against skidding decreases with increasing design speed.

Comparison of Tables 2 and 3 indicates that rollover is a particular concern for trucks. Under the assumed design conditions for horizontal curves, a truck will roll over before it will skid on a dry pavement. Under the assumed design conditions on a wet pavement, a truck will roll over before it skids at design speeds of 64 km/hr (40 mph) and below; above that speed, a truck will skid before it rolls over.

Deviations from Assumed Design Conditions

The margins of safety against skidding and rollover are a measure of the extent to which real-world drivers, vehicles, and highways can deviate from the assumed conditions without resulting in a skid or a rollover. Deviations from assumed conditions that can increase the likelihood of skidding include:

- Vehicles traveling faster than the design speed.
- Vehicles turning more sharply than the curve radius (oversteering).
- Lower pavement friction than assumed by AASHTO.
- Poorer tires than assumed by AASHTO.

Traveling faster than the design speed and turning more sharply than the curve radius would also increase the likelihood of rollovers. The likelihood of rollover would also be increased for a truck with a rollover threshold less than the assumed value of 0.30 g.

It would seem logical that the practice of providing less than full superelevation at the point of curvature (PC) would also increase the likelihood of rollovers, but this is not necessarily the case. Horizontal curves without spiral transitions are typically designed with two-thirds of the superelevation runoff on the tangent in advance of the PC and one-third of the superelevation runoff on the curve itself. Thus only two-thirds of the design superelevation is available at the PC, and this lack of full superelevation at the PC would appear to have the potential to offset up to approximately 0.30 g of the available margin of safety. However AASHTO policy assumes and field and simulation studies (for passenger cars) confirm that even on horizontal curves without spiral transitions, drivers tend to steer a spiral path. Thus when maximum superelevation is not available, the driver is usually not steering a minimum-radius path.

Computer simulation studies of trucks traversing horizontal curves (*2*) have found that the development of full superelevation on the tangent approach to a conventional circular curve actually results in slightly more lateral acceleration than development of superelevation with the two-thirds–one-third rule. Although the difference in lateral acceleration is small—at most 0.03 g—it is in the wrong direction, so development of full superelevation on the tangent is not a desirable approach to reducing truck rollovers. The same study found a small decrease in lateral acceleration—typically less than 0.01 g—when spiral transitions rather than the two-thirds–one-third rule were used to develop the superelevation. Thus the use of spiral transitions is desirable, but because of the small reduction in lateral acceleration, the use of spirals is unlikely to provide a major reduction in rollover accidents.

Field data for passenger cars and simulation results for trucks show that vehicles traversing a curve do not precisely follow the curve (*2,4*). Thus, although the path may have a larger radius than the curve at the PC, it will also have a smaller radius than the curve at some point in the curve. Simulation results show that the maximum lateral acceleration occurs several hundred feet after entering a curve. However, simulation results also show that the maximum deviation of lateral acceleration above the value obtained from the standard curve formula is approximately 0.02 g, which would offset a small portion of the margins of safety against rollover and skidding (*2*). Field studies for passenger cars suggest that this is a reasonable average value, but more extreme values can occur. Truck drivers may follow the curve more closely than passenger car drivers, but there are no data on this issue.

The review of the potential for safety problems created by deviations from the design assumptions indicates that traveling faster than the design speed of the curve is the single greatest concern. This is a particular concern on freeway ramps for two reasons. First, freeway ramps generally have lower design speeds than mainline roadways, which means that they have lower margins of safety against rollover (but higher margins of safety against skidding). Second, vehicles are especially likely to travel faster than the design speed on off-ramps, where vehicles traveling at higher speeds enter the ramp from the mainline roadway.

Table 4 compares the speeds at which skidding or rollover would occur for passenger cars and trucks traversing minimum-radius curves designed in accordance with current AASHTO cri-

TABLE 3 Margins of Safety Against Rollover on Horizontal Curves (2)

		Passenger car			Truck					
							Rollover margin of safety			
Design speed (mph)	Maximum e	Maximum comfortable lateral acceleration	Minimum radius (ft)	Rollover margin of safety RT = 1.20 g	Maximum comfortable lateral acceleration	Minimum radius (ft)	RT = 0.27	RT = 0.30	RT = 0.35	RT = 0.40
20	0.04	0.17	127	1.03	0.17	127	0.10	0.13	0.18	0.23
30	0.04	0.16	302	1.04	0.16	302	0.11	0.14	0.19	0.24
40	0.04	0.15	573	1.05	0.15	573	0.12	0.15	0.20	0.25
50	0.04	0.14	955	1.06	0.14	955	0.13	0.16	0.21	0.26
60	0.04	0.12	1,528	1.08	0.12	1,528	0.15	0.18	0.23	0.28
20	0.06	0.17	116	1.03	0.17	116	0.10	0.13	0.18	0.23
30	0.06	0.16	273	1.04	0.16	273	0.11	0.14	0.19	0.24
40	0.06	0.15	509	1.05	0.15	509	0.12	0.15	0.20	0.25
50	0.06	0.14	849	1.06	0.14	849	0.13	0.16	0.21	0.26
60	0.06	0.12	1,348	1.08	0.12	1,348	0.15	0.18	0.23	0.28
70	0.06	0.10	2,083	1.10	0.10	2,083	0.17	0.20	0.25	0.30
20	0.08	0.17	107	1.03	0.17	107	0.10	0.13	0.18	0.23
30	0.08	0.16	252	1.04	0.16	252	0.11	0.14	0.19	0.24
40	0.08	0.15	468	1.05	0.15	468	0.12	0.15	0.20	0.25
50	0.08	0.14	746	1.06	0.14	746	0.13	0.16	0.21	0.26
60	0.08	0.12	1,206	1.08	0.12	1,206	0.15	0.18	0.23	0.28
70	0.08	0.10	1,910	1.10	0.10	1,910	0.17	0.20	0.25	0.30
20	0.10	0.17	99	1.03	0.17	99	0.10	0.13	0.18	0.23
30	0.10	0.16	231	1.04	0.16	231	0.11	0.14	0.19	0.24
40	0.10	0.15	432	1.05	0.15	432	0.12	0.15	0.20	0.25
50	0.10	0.14	694	1.06	0.14	694	0.13	0.16	0.21	0.26
60	0.10	0.12	1,091	1.08	0.12	1,091	0.15	0.18	0.23	0.28
70	0.10	0.10	1,637	1.10	0.10	1,637	0.17	0.20	0.25	0.30

TABLE 4 Vehicle Speed at Impending Skidding or Rollover on Horizontal Curves for AASHTO High-Speed Design Criteria (*2*)

Design speed (mph)	Maximum e	Maximum comfortable lateral acceleration	Minimum radius (ft)	Passenger car available cornering f	Passenger car speed (mph)			Truck speed (mph)					
					At impending skid (wet)	At impending skid (dry)	At rollover RT = 1.20 g	At impending skid (wet)	At impending skid (dry)	At rollover			
										RT = 0.27 g	RT = 0.30 g	RT = 0.35 g	RT = 0.40 g
20	0.04	0.17	127	0.58	34.4	43.3	48.6	27.9	34.9	24.3	25.4	27.3	29.0
30	0.04	0.16	302	0.51	49.8	66.7	74.9	40.5	53.8	37.5	39.2	42.0	44.6
40	0.04	0.15	573	0.46	65.8	91.9	103.2	53.7	74.2	51.6	54.1	57.9	61.5
50	0.04	0.14	955	0.44	82.5	118.6	133.3	67.4	95.7	66.6	69.8	74.7	79.4
60	0.04	0.12	1,528	0.42	102.7	150.1	168.6	84.0	121.1	84.3	88.3	94.5	100.4
20	0.06	0.17	116	0.58	33.4	41.8	46.8	27.3	33.9	24.0	25.0	26.7	28.3
30	0.06	0.16	273	0.51	48.2	64.1	71.8	39.6	52.0	36.8	38.4	41.0	43.4
40	0.06	0.15	509	0.46	63.3	87.5	98.1	52.1	71.0	50.2	52.4	55.9	59.3
50	0.06	0.14	849	0.44	79.4	113.0	126.7	65.5	91.7	64.8	67.7	72.3	76.5
60	0.06	0.12	1,348	0.42	98.6	142.4	159.6	81.4	115.5	81.7	85.3	91.1	96.4
70	0.06	0.10	2,083	0.41	120.7	177.0	198.4	99.7	143.6	101.5	106.1	113.2	119.9
20	0.08	0.17	107	0.58	32.5	40.5	45.3	26.8	33.0	23.7	24.7	26.3	27.8
30	0.08	0.16	252	0.51	47.0	62.2	69.6	39.0	50.7	36.4	37.9	40.3	42.6
40	0.08	0.15	468	0.46	61.8	84.7	94.8	51.3	69.1	49.6	51.6	54.9	58.0
50	0.08	0.14	764	0.44	76.8	108.2	121.1	63.9	88.3	63.3	66.0	70.2	74.2
60	0.08	0.12	1,206	0.42	95.2	136.0	152.2	79.3	110.9	79.6	82.9	88.2	93.2
70	0.08	0.10	1,910	0.41	118.0	171.2	191.5	98.5	139.6	100.1	104.3	111.0	117.3
20	0.10	0.17	99	0.58	31.8	39.3	43.9	26.4	32.2	23.4	24.4	25.9	27.2
30	0.10	0.16	231	0.51	45.9	60.1	67.1	38.3	49.2	35.8	37.2	39.5	41.6
40	0.10	0.15	432	0.46	60.5	82.2	91.8	50.6	67.3	49.0	50.9	54.0	56.9
50	0.10	0.14	694	0.44	74.6	104.2	116.3	62.6	85.4	62.1	64.5	68.4	72.1
60	0.10	0.12	1,091	0.42	92.3	130.6	145.9	77.6	107.0	77.8	80.9	85.8	90.5
70	0.10	0.10	1,637	0.41	111.5	160.0	178.7	93.8	131.1	95.3	99.1	105.1	110.8

TABLE 5 Lateral Acceleration Developed by Overdriving Design Speed for Horizontal Curves Designed to AASHTO Minimum Radii for High-Speed Design (2)

Design speed (mph)	Maximum superelevation (e_{max})	Maximum comfortable lateral acceleration	Minimum radius of curvature (ft)	Side friction demand for overdriving design speed of curve by:				
				0 mph	5 mph	10 mph	15 mph	20 mph
20	0.04	0.17	127	0.17	0.29	0.43	0.60	0.80
30	0.04	0.16	300	0.16	0.23	0.32	0.41	0.52
40	0.04	0.15	561	0.15	0.20	0.26	0.32	0.39
50	0.04	0.14	926	0.14	0.18	0.22	0.26	0.31
60	0.04	0.12	1,500	0.12	0.15	0.18	0.21	0.24
20	0.06	0.17	116	0.17	0.30	0.46	0.64	0.86
30	0.06	0.16	273	0.16	0.24	0.33	0.43	0.55
40	0.06	0.15	508	0.15	0.21	0.27	0.34	0.41
50	0.06	0.14	833	0.14	0.18	0.23	0.28	0.33
60	0.06	0.12	1,333	0.12	0.15	0.18	0.22	0.26
65	0.06	0.11	1,657	0.11	0.14	0.17	0.20	0.23
70	0.06	0.10	2,042	0.10	0.13	0.16	0.18	0.20
20	0.08	0.17	107	0.17	0.31	0.48	0.69	0.92
30	0.08	0.16	250	0.16	0.25	0.35	0.46	0.59
40	0.08	0.15	464	0.15	0.21	0.28	0.35	0.44
50	0.08	0.14	758	0.14	0.19	0.24	0.29	0.35
60	0.08	0.12	1,200	0.12	0.15	0.19	0.23	0.28
65	0.08	0.11	1,482	0.11	0.14	0.17	0.21	0.24
70	0.08	0.10	1,815	0.10	0.13	0.16	0.19	0.22
20	0.10	0.17	99	0.17	0.32	0.51	0.73	0.98
30	0.10	0.16	231	0.16	0.25	0.36	0.49	0.62
40	0.10	0.15	427	0.15	0.22	0.29	0.37	0.46
50	0.10	0.14	694	0.14	0.19	0.25	0.31	0.37
60	0.10	0.12	1,091	0.12	0.16	0.20	0.24	0.29
65	0.10	0.11	1,341	0.11	0.14	0.18	0.22	0.26
70	0.10	0.10	1,633	0.10	0.13	0.16	0.19	0.23

teria. These speeds are computed by setting the value of f in Equation 3 equal to the available pavement friction coefficient or rollover threshold and solving for the vehicle speed (V). Table 4 shows that on a dry pavement a passenger car will skid at a lower speed than it will roll over, and a truck with a rollover threshold of 0.30 g will roll over at a lower speed than it will skid. On a wet pavement, a passenger car will still skid at a lower speed than it will roll over. However a truck will skid before it will roll over at design speeds of 64 km/hr (40 mph) or less under the assumed values for wet pavement conditions. If a wet pavement has above-minimum friction, however, the truck may still roll over at a lower speed than it will skid. Finally for horizontal curve design speeds over 64 km/hr (40 mph), the truck will always roll over before it will skid under the assumed design conditions.

Table 5 presents the results of an alternative sensitivity analysis that shows the lateral accelerations that result from overdriving horizontal curves at speeds of up to 32 km/hr (20 mph) above the design speed. Table 5 addresses curves designed to the AASHTO minimum radius for specified values of design speed and maximum superelevation rate. Curves designed with larger radii than the AASHTO minimum will produce lower lateral accelerations than those shown in Table 5. The results shown in Table 5 are in accord with operational experience. At lower design speeds, overdriving of the design speed by even a small amount can produce side friction demands above the rollover thresholds of some trucks. On the other hand, at higher design speeds, overdriving of the design speed by as much as 32 km/hr (20 mph) does not produce enough lateral acceleration to produce a truck rollover.

LOW-SPEED HORIZONTAL CURVE DESIGN FOR INTERSECTIONS AND TURNING ROADWAYS

The low-speed horizontal curve design criteria presented in Green Book Table III-17 are intended for use at intersections and turning roadways with design speeds of 64 km/hr (40 mph) or less. Harwood and Mason (*9*) have presented an evaluation of the situations in which the low-speed criteria in contrast to the high-speed or open-highway criteria should be used.

The low-speed design criteria are based on higher values of maximum side friction demand (f_{max}) than can be used in high-speed design. A comparison of the permitted values of f_{max} is presented in Table 6. At design speeds of 32 and 48 km/hr (20 and 30 mph), substantially more side friction demand is permitted under the low-speed design criteria than under the high-speed design criteria.

Green Book Table III-17 is based on an assumed *minimum* superelevation rate for each design speed rather than a user-selected *maximum* superelevation rate. The table shows specified values of minimum radius, although it is not clear how a minimum radius can be computed from a maximum side friction factor and a *minimum* (rather than a maximum) superelevation rate. Green Book Table IX-12 presents the range of superelevation rates permitted for particular horizontal curve radii. The Green Book does not make clear whether horizontal curve radii less than those specified in Table III-17 can be used when higher-than-minimum superelevation rates are used. In high-speed design, increasing the maximum superelevation rate (e_{max}) decreases the minimum radius for a horizontal curve.

TABLE 6 Comparison of Permitted Values of f_{max}

Design Speed (mph)	High-Speed Design (Table III-6)	Low-Speed Design (Table III-17)
10	—	0.38
20	0.17	0.27
30	0.16	0.20
40	0.15	0.16
50	0.14	—
60	0.12	—
70	0.10	—

Table 7 compares the vehicle speed for passenger cars and trucks at impending skidding and rollover for design speeds of from 16 to 48 km/hr (10 to 40 mph). Table 7 compares vehicle speeds at impending skidding and rollover for

- High-speed design on the basis of Green Book Table III-6.
- Low-speed design on the basis of the minimum radii in Green Book Table III-17.
- Low-speed design for radii equal to the lesser of the minimum radii in Table III-17 and the radii calculated from Equation 3 by using the values of f_{max} specified in Table III-17.

All other assumptions concerning vehicle and pavement characteristics remain the same as in earlier analyses.

For horizontal curves designed in accordance with the minimum radii specified in Green Book Table III-17, there do not appear to be any critical skidding or rollover problems for passenger cars. However Table 7 shows that in every case for design speeds of 16 and 32 km/hr (10 and 20 mph) a truck could skid or roll over by exceeding the design speed of a minimum-radius curve by 6.4 km/hr (4 mph) or less. This analysis suggests that the low-speed design criteria in Green Book Table III-17 may not be adequate to safely accommodate some trucks in very critical situations.

As discussed above, the Green Book does not make clear whether, for low-speed design, it is permissible to use a smaller-curve radius than that shown in Table III-17 if an above-minimum superelevation rate is used. However, Table 7 shows that on curves with a 16-km/hr (10-mph) design speed, trucks could skid—and the most unstable trucks could roll over—at speeds less than the design speed. On curves with a 32-km/hr (20-mph) design speed, the most unstable trucks could roll over when traveling at less than 1.6 km/hr (1 mph) above the design speed.

Low-speed design criteria for urban streets presented in Green Book Table III-16 are based on assumed values of f_{max} that are slightly higher even than those in Table III-17. Thus, the same issues discussed above with respect to Green Book Table III-17 are an even greater concern with respect to Table III-16.

CONCLUSIONS AND RECOMMENDATIONS

High-Speed Horizontal Curve Design Criteria

The following conclusions and recommendations have been drawn from the results presented in this paper concerning the AASHTO high-speed or open-highway horizontal curve design criteria presented in Green Book Table III-6.

1. On horizontal curves designed in accordance with AASHTO high-speed criteria, a passenger car with poor tires on a poor wet

TABLE 7 Vehicle Speed at Impending Skidding or Rollover on Horizontal Curves for AASHTO High-Speed and Low-Speed Design Criteria

		High-speed design					Low-speed design									
		Minimum radius as specified in Table III-6					Minimum radius as specified in Table III-17					Minimum radius calculated from maximum f in Table III-17 or minimum radius in Table III-17 (if smaller)				
			Passenger car		Truck			Passenger car		Truck			Passenger car		Truck	
Design speed (mph)	Maximum e	Radius (ft)	Speed at impending skid	Speed at impending rollover	Speed at impending skid	Speed at impending rollover	Radius (ft)	Speed at impending skid	Speed at impending rollover	Speed at impending skid	Speed at impending rollover	Radius (ft)	Speed at impending skid	Speed at impending rollover	Speed at impending skid	Speed at impending rollover
Maximum e = 0.02																
10	0.02	--	--	--	--	--	25	15.9	21.4	12.8	11.0	17	13.0	17.5	10.4	8.9
20	0.02	--	--	--	--	--	90	28.5	40.6	22.9	20.8	90	28.5	40.6	22.9	20.8
30	0.02	--	--	--	--	--	230	42.7	64.9	34.4	33.2	230	42.7	64.9	34.4	33.2
40	0.02	--	--	--	--	--	430	55.9	88.7	45.1	45.4	430	55.9	88.7	45.1	45.4
Maximum e = 0.04																
10	0.04	--	--	--	--	--	25	16.1	21.6	13.1	11.3	16	12.8	17.2	10.4	9.0
20	0.04	127	34.4	48.6	27.9	25.4	90	28.9	40.9	23.5	21.4	86	28.3	40.0	23.0	20.9
30	0.04	302	49.8	74.9	40.5	39.2	230	43.5	65.4	35.4	34.2	250	43.5	65.4	35.4	34.2
40	0.04	573	65.8	103.2	53.7	54.1	430	57.0	89.4	46.5	46.8	430	57.0	89.4	46.5	46.8
Maximum e = 0.06																
10	0.06	--	--	--	--	--	25	16.3	21.7	13.3	11.6	15	12.7	16.9	10.4	9.0
20	0.06	116	33.4	46.8	27.3	25.0	90	29.4	41.2	24.1	22.0	81	27.9	39.1	22.8	20.9
30	0.06	273	48.2	71.8	39.6	38.4	230	44.2	65.9	36.3	35.2	230	44.2	65.9	36.3	35.2
40	0.06	509	63.3	98.1	52.1	52.4	430	58.1	90.1	47.9	48.2	430	58.1	90.1	47.9	48.2
Maximum e = 0.08																
10	0.08	--	--	--	--	--	25	16.6	21.9	13.6	11.9	14	12.6	16.7	10.4	9.1
20	0.08	107	32.5	45.3	26.8	24.7	90	29.8	41.6	24.6	22.6	76	27.5	38.2	22.7	20.8
30	0.08	252	47.0	69.6	39.0	37.9	230	45.0	66.5	37.3	36.2	214	43.5	64.1	36.0	34.9
40	0.08	468	61.8	94.8	51.3	51.6	430	59.2	90.9	49.2	49.5	430	59.2	90.9	49.2	49.5
Maximum e = 0.10																
10	0.10	--	--	--	--	--	25	16.0	22.1	13.3	12.2	14	11.9	16.5	9.9	9.1
20	0.10	99	31.8	43.9	26.4	24.4	90	30.3	41.9	25.2	23.2	72	27.1	37.5	22.5	20.8
30	0.10	231	45.9	67.1	38.3	37.2	230	48.4	67.0	40.2	37.1	200	45.2	62.4	37.5	34.6
40	0.10	432	60.5	91.8	50.6	50.9	430	66.2	91.6	55.0	50.8	410	64.7	89.4	53.7	49.6

NOTE: Shaded areas are not permitted by Green Book Table III-17 which specifies minimum superelevation.

pavement will generally skid at a lower speed than it will roll over. However, even minimum-radius curves designed in accordance with AASHTO policy provide an adequate margin of safety against both vehicle skidding and rollover for passenger cars traveling at the design speed.

2. On minimum-radius curves designed in accordance with AASHTO high-speed criteria, the most unstable trucks (i.e., those with the highest centers of gravity) will roll over before they will skid off the road on a dry pavement. However, on a poor wet pavement, a truck with poor tires on a minimum-radius curve will generally skid at a lower speed than it will roll over on curves with design speeds of up to 64 km/hr (40 mph). For horizontal-curve design speeds above 64 km/hr (40 mph), the most unstable trucks will roll over at a lower speed than they will skid off the road.

3. The margins of safety against skidding and rollover by trucks appear to be adequate for trucks that do not exceed the design speed on curves designed in accordance with Green Book Table III-6.

4. Variations in the methods for developing superelevation on horizontal curves, such as the provision of spiral transitions, have only very small effects on the likelihood of skidding or rolling over by trucks.

5. On horizontal curves with lower design speeds that are designed in accordance with Green Book Table III-6, the most unstable trucks can roll over when traveling as little as 8 to 16 km/hr (5 to 10 mph) above the design speed. This is a particular concern on freeway ramps, many of which have unrealistically low design speeds in comparison with the design speed of the mainline roadway. A recent paper by Harwood and Mason (*9*) reviews the existing AASHTO criteria for selecting the design speed of a ramp as it relates to the highway design speeds. The selection of realistic design speeds is critical to safety, particularly for trucks.

On the basis of these evaluation results there does not appear to be a need to modify existing criteria for determining the radii and superelevations of horizontal curves in Green Book Table III-6. Existing design policies provide adequate margins of safety against skidding and rollover by both passenger cars and trucks as long as the design speed of the curve is selected realistically. Special care should be taken for curves with design speeds of 48 km/hr (30 mph) or less to ensure that the selected design speed will not be exceeded, particularly by trucks. Design of superelevation transitions according to the two-thirds–one-third rule provides an acceptable design, although spiral transitions would provide marginally lower lateral accelerations.

Low-Speed Horizontal Curve Design Criteria

AASHTO policy permits the low-speed design criteria presented in Green Book Table III-17 to be used for horizontal curves at intersections and turning roadways with design speeds of 64 km/hr (40 mph) or less. The following conclusions and recommendations were drawn from the evaluation of these low-speed design criteria.

1. Minimum-radius horizontal curves designed in accordance with the low-speed criteria in Green Book Table III-17 generally provide adequate margins of safety against skidding and rollover for passenger cars traveling at the design speed.

2. For design speeds of 16 to 32 km/hr (10 to 20 mph), minimum-radius horizontal curves may not provide adequate margins of safety for trucks with poor tires on a poor wet pavement or for trucks with low rollover thresholds. Revision of the criteria in Green Book Table III-17 should be considered, especially for locations with substantial truck volumes. This same concern is applicable to the horizontal curve design criteria for low-speed urban streets based on Green Book Table III-16.

3. The Green Book should be revised to state explicitly that minimum radii smaller than those shown in Table III-17 should not be used, even when they appear justified by above-minimum superelevation rates.

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Safety Aspects of Individual Design Elements and Their Interactions on Two-Lane Highways: International Perspective

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The results of an extensive literature review of the safety performances of low- and intermediate-traffic-volume, two-lane rural highways are presented. The effects on traffic safety, as measured by accident rates, of pavement width, radius of curve/degree of curve, gradient, sight distance, traffic volume, and design speed on curved sections of two-lane rural highways are covered. The following are some of the main findings. There is a distinct tendency for accidents to decrease with increasing pavement width up to about 7.5 m (25 ft). There exists a negative relationship between radius of curve and accident rate. The sharper the radius of curve, the higher the number of run-off-the-road accidents. Curves that dictate a significant change in operating speeds and that cause nonhomogeneity in road characteristics are especially dangerous. The most successful parameter in explaining the variability in accident rates was degree of curve (United States) or curvature change rate (Europe). Gradients of up to about 6 percent have a relatively small effect on the accident rate. A sharp increase in the accident rate was noted on grades of more than 6 percent. There exists a negative relationship between available sight distance and accident risk. However, other influencing parameters, such as wide pavements and gentle radii of curve, might also play a part in the observed positive effect of sufficient sight distances on the accident situation. For narrow road sections, an increase in sight distance could favorably affect traffic safety. A negative relationship between traffic volume and road traffic accidents was established. Run-off-the-road accidents were found to decrease with increasing average annual daily traffic up to 10,000 vehicles per day. Recent investigations reported a U-shaped distribution between accident rate and traffic volume. An accident rate of 2.0 accidents per 10^6 vehicle km (3.2 accidents per 10^6 vehicle mi) was proposed as a breakpoint between levels of safety and unsafety. This breakpoint was derived from relationships between single design parameters and accident rate as well as from the superimposition of the design parameters. Limiting values for a number of design parameters are also proposed. If these limiting values were exceeded, the proposed breakpoint, in relation to the accident rate, would be exceeded.

"Two-lane rural highway safety is an issue of pressing national concern. It has been identified as the highest priority research need by the Transportation Research Board's Committee on Geometric Design (A2A02)" (*1*).

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The approximately 5.0 million km (3.1 million mi) of two-lane rural highways in the United States represents 97 percent of rural mileage and 80 percent of all U.S. highway miles. Two-lane rural highway travel constitutes an estimated 66 percent of rural highway travel and 30 percent of all U.S. highway travel. Two-lane rural highways have (*2*)

- Higher accident rates than all other kinds of rural highways except four-lane undivided roads.
- Higher percentages of head-on collisions than any other kind of rural highways and also higher percentages of single-vehicle accidents.

The probability of an accident on two-lane rural highways is highest at horizontal curves, intersections, and bridges (*2*).

Many factors may exhibit a measurable influence on driving behavior and traffic safety on two-lane rural highways. These include, but are not limited to,

1. Human factors, such as improper judgment of the road ahead and traffic, speeding, driving under the influence of alcohol or drugs, driving inexperience (young people), handicaps (especially for the older segment of the driving population), and sex (*3,4*).
2. Physical features of the site, such as horizontal and vertical alignments, and cross section combined with the degree of roadside development and access control.
3. Presence and action of traffic, such as traffic volume, traffic mix, and seasonal and daily variations.
4. Legal issues, such as overall mandatory federal and state laws, type of traffic control devices at the sites, and degree of enforcement.
5. Environmental factors, such as weather and pavement conditions.
6. Vehicle deficiencies, such as tires, brakes, and vehicle age.

All of the above therefore constitute a complex mix of various causes of traffic accidents, of which the road itself represents only one factor, but a very important one.

To show to what extent safety in traffic is influenced by the road itself, the first step would be to select those elements that may well characterize the latter. These include, most important, the design parameters, the cross section, and traffic volume, since they can easily be evaluated in terms of size and number. However, these parameters affect the accident situation collectively rather than independently. Therefore, if conclusions were to be made about the design and traffic conditions of the road with regard to traffic safety, it is necessary to consider these interdependencies. Investigations into the relationship between one or a combination of design and traffic volume parameters and the accident situation may give valuable results, as long as it is understood that these parameters are among a variety of influencing factors that are related.

Numerous quantitative and qualitative analyses, appraisals, and discussions of traffic safety have appeared in the literature of highway and traffic engineering to provide a better understanding of accident risk characteristics. In the planning, design, and operation of a highway transportation system, knowledge of such characteristics is imperative if sound engineering decisions are to be made (*5–8*).

In this paper an extensive international literature review, coupled with the results of research studies by the authors, was conducted to provide current information on the safety performance of two-lane rural highways. The study covered the effects on traffic safety, as measured by accident rates, of pavement width, radius of curve/degree of curve, gradient, sight distance, traffic volume, and design speed on two-lane rural highways. These geometric design parameters were chosen for analysis because

1. It was anticipated that they would exhibit a measurable influence,
2. They can easily be measured, and
3. Accident research studies found statistically measurable impacts of these parameters on traffic safety.

It should be noted that this review may not be totally comprehensive and complete. For instance, the authors would have liked to learn more about the sample sizes and statistical techniques used in the various research studies to give some valuable comments on the papers investigated or to evaluate the worth of the reviewed studies. Unfortunately this information was not available or was incomplete in many publications. It appears that until the 1970s there was a tendency to report only the results of research studies without sufficiently describing the data bases or the analysis techniques used. Today, a research paper that does not give information on exact sample sizes and the analysis techniques used is not likely to be accepted by the research community. For these reasons, the reader should understand that the aim of the paper is, to a certain extent, informative rather than critical.

BACKGROUND

By and large, most geometric design guidelines are based on driving dynamic considerations. For example, some European guidelines (*9–12*) regard design speed as a driving dynamic safety parameter and attempt to tune design speed with actual operating speed as an indirect safety criterion to provide safe and gentle curvilinear alignments.

Geometric design guidelines have long been the subject of dispute. Some argue that the guidelines do not present a clear measure for evaluating the safety level of roadways. For instance, in their discussion of the German design guidelines, Feuchtinger and Christoffers (*13*) stated, "When a road goes into operation, the accident experience afterwards is the only indicator of the safety performance of the road. During the planning stage, there is no way to tell what level there is for traffic safety." Similarly, Bitzl (*14*) stated, "Unlike other engineering fields, in road design it is almost impossible to determine the safety level of a road. In other words, the guidelines provide no basic values to describe the safety level of a road, in relation to design parameters and traffic conditions; whereas in other engineering fields, such as structural, there exist safety values for constructing, for example, bridges or buildings." Similar statements about safety levels in highway geometric design guidelines were made by Auberlen (*15*) and Krebs (*16*).

In a discussion of the German design guidelines (*9*), Krebs and Kloeckner (*8*) said,

> • If the guidelines guarantee the safety of a road, then "no" or "only few" accidents should occur on that road. When accidents happen, drivers are always the ones who take the blame for the mishap.
> • Accidents are not uniformly distributed on the road network. High accident locations are clear indication that, besides driver's error, there exist other influencing parameters which are characterized by the road itself. (*8*)

Along the same line, Mackenrot stated the following in a previous publication (*17*):

> • No one is in a position to state whether a driver's discipline was in order before a high accident location, but then failed at that location. When a driver fails at a high accident location, it is often said that it was his way of driving which caused the accident.
> • When a driver fails a number of times at certain locations, then it becomes obvious that the problem lies, not with the driver, but mainly with the geometry of the road itself. (*17*)

The above statements indicate that no one is in a position to state whether a road section of considerable length is safe or not, nor can anyone guarantee that a road section will provide a minimum level of safety or a maximum level of endangerment, that is, unsafety.

INFLUENCE OF SINGLE DESIGN PARAMETERS

Pavement Width

Figure 1(*a*) depicts the relationship between accident rate and pavement width as derived from the results of several research studies.

Research studies have generally shown that adequate pavement widths are necessary for safe driving operation. The necessary widths are generally the result of the dimension of design vehicles and lateral clearances for transportation and safety maneuvers. If these widths are not sufficiently designed, impairment of traffic safety can originate. Therefore, it can be expected that there exists a certain correlation between pavement width and traffic safety.

Baldwin (*18*) investigated accidents on rural two-lane highways in the United States. He indicated that the accident rate decreases as pavement width increases. On the basis of his study, it appears that

1. Pavement widths of less than 5.5 m (18 ft) create unfavorable conditions for traffic safety.
2. The gain in safety is relatively small for pavement widths greater than 7 m (23 ft).

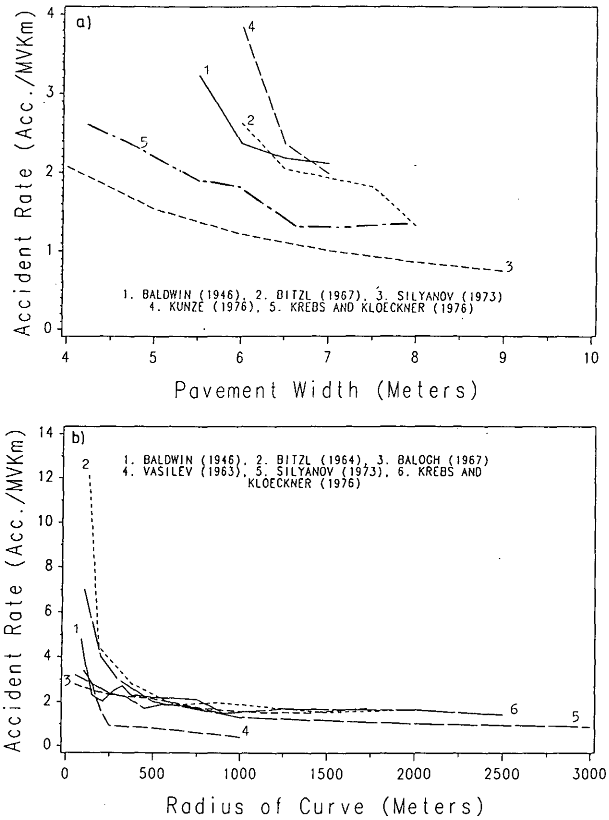


FIGURE 1 Examples illustrating the relationship between accident rate [number of accidents per 10^6 vehicle kilometers (Acc./MVkm)] and (*a*) pavement width and (*b*) radius of curve.

Cope (*19*) studied the effect of lane widening on accident rates on two-lane roadways in Illinois. He investigated 22 sections with an overall length of about 395 km (246 mi) whose pavements were widened from 5.5 m (18 ft) to 6.7 m (22 ft). The results of investigations done before and after the widening showed that the increase in pavement width reduced the accident rate from 1.4 to 0.9 accidents per 10^6 vehicle km (2.3 to 1.4 accidents per 10^6 vehicle mi). He also reported that the largest decrease in accident rate was on sections that had a high accident rate in the investigation done before the widening.

Bitzl (*20*) found a marked negative relationship between accident rate and pavement width in the Federal Republic of Germany (FRG). In several of his later investigations, Bitzl confirmed this result. For instance, in 1967 he stated the following: "Such a relationship is understandable since, if wider lane widths were available, overtaking or passing maneuvers could be accomplished with greater ease and smaller degree of danger" (*21*).

An investigation done by Winch (*22*) in Canada indicated that an increase in lane width leads to a decrease in accident frequency on two-lane rural roads. Similar conclusions were reported by Balogh (*23*) for Hungarian roads.

A comprehensive evaluation of international results by Silyanov in the former USSR (*24*) revealed that the accident rate decreases as pavement width increases for pavement widths of between 4 m (13 ft) and 9 m (30 ft). On wide pavements, he indicated that the accident rate decreases at a much slower pace than on narrower pavements.

A study cited by Pignataro (*25*) showed that the total accident rate per 10^6 vehicle mile decreased from 5.5 to 2.4 as the pavement width increased from 5 m (16 ft) to 7.5 m (25 ft). Another study, also cited by Pignataro (*25*), covering about 385 km (240 mi) of highways that had been widened from 5.5 m (18 ft) to 6.7 m (22 ft), indicated that the accident rate reduction ranged from 21.5 percent for low-volume roads to 46.6 percent for high-volume roads.

Kunze (*26*) studied the relationship between accident rate and pavement width classes in the FRG. He established a clear tendency for the accident rate to decrease with increasing pavement width classes for all accidents: run-off-the-road accidents, accidents at intersections, and head-on and rear-end accidents. Krebs and Kloeckner (*8*) established a negative linear relationship between pavement width and accident risk on two-lane rural highways. Zegeer et al. (*27*) reported that the accident rate in the United States decreases as pavement width increases up to about 7.25 m (24 ft).

A study by McCarthy et al. (*28*) on the effects of widening of lanes at 17 sites, in which the lanes were widened from 2.7 and 3.0 m (9 and 10 ft) to 3.4 and 3.7 m (11 and 12 ft), showed that a lane width increase reduced the injury-fatality accident rate significantly (22 percent) and caused a decrease in the total accident rate.

In addition to pavement width, Cirillo and Council (*29*) reported the following about shoulder width in the United States: "Most studies agree that shoulders up to 1.8 m (6 ft) wide on facilities with greater than 1000 ADT provide a safety benefit. The effect beyond 1.8 m (6 ft) is not clear."

Choueiri (*5*) and Lamm and Choueiri (*6,7*), who studied the joint impact of several parameters—degree of curve, length of curve, superelevation rate, gradient up to 5 percent, sight distance, lane width, shoulder width, and average annual daily traffic (AADT)—on accident rates on 261 two-lane curved sections in New York State, established a marginal negative relationship between pavement width and accident rate. The pavement widths considered in their study were 6.1 m (20 ft), 6.7 m (22 ft), and 7.3 m (24 ft). Shoulder width did not have a significant effect on the accident rate.

Statistical analyses by Zegeer et al. (*30*) of accident relationships based on an analysis of 10,900 horizontal curves on two-lane rural highways in Washington State with corresponding accident (12,123 accidents), geometric, traffic, and roadway data variables determined a 21 percent accident rate reduction for 1.2 m (4 ft) of lane widening.

The research studies reported above have generally shown that accident rates decrease with an increase in pavement width of up to about 7.5 m (25 ft) on two-lane rural roads. This increase in traffic safety was evident for all classes of radii of curves, gradients, and traffic volumes.

An extensive before-and-after investigation (*31*) covering 3 years of accident experience (1,428 accidents) on 28 sections with a section length of about 90 km (55 mi) redesigned according to the German design guidelines (*9*) established the results given in Figure 2 between the accident rate-accident cost rate and the pavement width. The regression curves in Figure 2 are based on the vehicle mileage calculated for every pavement width class. The relationship between accident rate and pavement width is in agreement with prior research; that is, the risk of being involved in an accident decreases as pavement width increases.

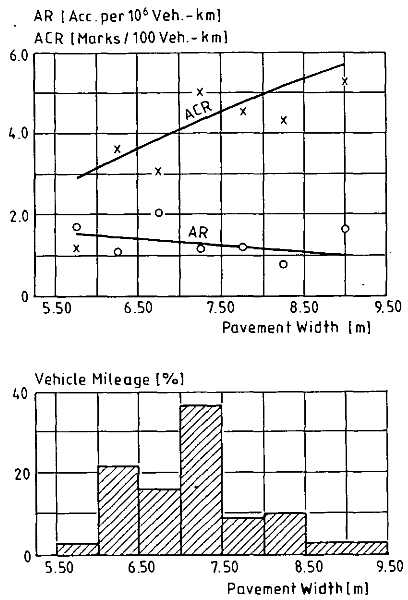


FIGURE 2 Accident rate (AR, in number of accidents per 10^6 vehicle kilometers) and accident cost rate (ACR, in Deutschmarks per 10^2 vehicle kilometers, where 1 DM $\simeq$ \$0.60, in 1994) versus pavement width.

Contrary to the opinions of many experts, Figure 2 shows that the accident cost rate, an indicator of accident severity, increases as pavement width increases, even though the investigated road sections were designed according to the German design guidelines. This result may be because wide pavements are usually assigned high design speed levels. With high design speeds, operating speeds are usually high. Consequently, as operating speeds increase, the severity of an accident, as measured by the accident cost rate, increases too.

Radius of Curve and Degree of Curve

Figure 1(*b*) shows the relationship between accident rate and the radius of the curve as derived from the results of several research studies.

The safe and efficient movement of traffic is greatly influenced by the geometric features of the highway. A review of accident spot maps normally shows that accidents tend to cluster on curves, particularly on very sharp curves. Even though the design engineer possesses detailed information—derived from driving dynamic formulas and standard values—on driving through a curve, accident frequency and severity often do not appear to coincide with the actual driving behavior. Recently, there have been attempts during the design stages to consider the expected operating speeds on curves. This is suggested by a number of researchers—such as Lamm (*32*), Leisch and Leisch (*33*), Koeppel and Bock (*34*), and Hayward et al. (*35*)—and is required in the German design guidelines (*9*) and the Swiss design standard (*11*).

The horizontal alignment of the road may not be characterized by radius of curve only. The same radius of curve in a sequence of similarly tuned radii of curve can have effects on the accident situation other than those in a nontuned sequence of different radii of curve, as is usually the case on most old alignments.

The general opinion today is that the accident risk decreases as the radius of the curve increases or as the degree of the curve decreases. However, different opinions exist regarding the extent of this influence on the accident situation. An investigation by Baldwin (*18*) of U.S. roads with traffic volumes of less than 5,000 vehicles per day indicated that the accident rate decreases as the radius of curve increases. Pfundt (*36*) studied accidents on low- and high-volume roads in the FRG. He indicated that sharp, low-volume roads had high accident frequencies. He also indicated that drivers tended to drive faster on low-volume roads than on high-volume roads. Baldwin (*18*) concluded that the accident rate decreases as curve frequency [radius of less than 600 m (1,970 ft)] increases. On the basis of his investigation, a single curve with a small radius should be regarded as more unfavorable than the same curve within a section with a sequence of curves with similar radii.

An investigation of injury accidents by Coburn (*37*) in the United Kingdom indicated that the accident rate was especially high on curves with radii of less than 175 m (580 ft). For curves with larger radii, he indicated that the increase in traffic safety was relatively small. In a later publication, Coburn (*38*) stated that the accident rate on sharp curves was higher than that on gentle curves.

Balogh (*23*) in Hungary, Raff (*39*) in the United States, Bitzl (*40*) in the FRG, and Vasilev (*41*) and Babkov (*42*) in the former USSR also indicated that the accident rate decreases as the radius of the curve increases.

Knoflacher (*43*) indicated that in the FRG, for radii of up to 800 m (2,625 ft), the percentage of skidding accidents on wet pavements was higher than that on dry pavements; for radii of less than 250 m (820 ft), he found the difference to be statistically significant.

Wilson (*44*) reported that the accident rate on curves with radii of less than 170 m (560 ft) was about five times that on curves with radii of greater than 910 m (2,990 ft). He pointed out the danger that a single curve after a long tangent poses. The dangers that single isolated curves pose was also mentioned by Babkov (*45*) in the former USSR. Because of speed differences before and within the curve, Babkov (*45*) spoke of

- "Safe curves," when the change in speeds was less than 20 percent;
- "Relatively safe curves," when the change in speeds was between 20 and 40 percent;
- "Dangerous curves," when the change in speeds was between 40 and 60 percent; and
- "Very dangerous curves," when the change in speeds was greater than 60 percent.

Pfundt (*36*) indicated that nearly two-thirds of run-off-the-road accidents in the FRG occur in curves or near curved sites. For road sections with different road characteristics, he indicated that the risk of undergoing run-off-the-road accidents increases with the increasing complexity of the alignment. He also indicated that road sections with few curves are more dangerous than sections with many curves.

From the accident data bases of a number of countries, Silyanov (*24*) in the former USSR established a distinct tendency for the accident rate to decrease with an increasing radius of curve. In a study of accidents in Great Britain, O'Flaherty (*46*) also concluded that the accident rate decreases as the radius of curve increases.

Krebs and Kloeckner (*8*) in the FRG determined the following:

1. Accident risk decreases with an increasing radius of curve.
2. Road sections with radii of less than 200 m (660 ft) have accident rates that are twice as high as those on sections with radii of greater than 400 m (1,300 ft).
3. A radius of 400 m provides a cross-point in safety.
4. For radii greater than 400 m, the gain in safety is relatively small.
5. For road sections with radii of between 500 m (1,600 ft) and 800 m (2,600 ft), a slight increase in accident risk is sometimes shown.

Lamm (*32*) explained Item 5 in the following manner:

> Large radii are often associated with low design speeds, such as 80 km/h (50 mph), for which corresponding superelevation rates are between 2 and 3 percent. However, actual 85th percentile speeds on these curves require superelevation rates of at least 5.5 percent. Such a discrepancy between design speed and actual operating speed could influence the accident situation unfavorably. (*32*)

Rumar (*47*) analyzed 14,000 accidents on 9000 km (5,595 mi) of two-lane roads in Sweden. His results showed a reduction in accident rates with increasing radii of horizontal curves.

Statistical analyses by Zegeer et al. (*30*) revealed

1. Significantly higher numbers of accidents on sharper curves, and
2. Accident reductions of up to 80 percent depending on the central angle and the amount of curve flattening.

The research studies reported above have generally shown that there exists a negative relationship between the radius of curve and the accident rate. A considerable increase in accident risk exists in particular on curves with sharp radii, where run-off-the-road accidents most frequently occur, especially after long tangents. In addition to the size of the radius of curve, the road characteristics play an important role. Curves that dictate a significant change in operating speeds and cause inconsistencies in road characteristics are especially dangerous (*1,48,49*). Furthermore, the pavement width influences to a certain extent the magnitude of the accident rate. Curves that are combined with wide pavements do not affect the accident risk as unfavorably as those curves that are combined with narrow pavements.

Research by the authors (*1,5–7,48,49*) demonstrated the following:

1. The most successful parameter in explaining the variability in accident rates was degree of curve.
2. For all lanes combined,
   a. Gentle curvilinear horizontal alignments consisting of tangents or transition curves combined with curves of up to 5 degrees showed the lowest average accident risk;
   b. Accident risk on sections with a change in curve of between 5 and 10 degrees was at least twice as high as that on sections with a change in curve of between 1 and 5 degrees;
   c. Accident risk on sections with a change in curve of between 10 and 15 degrees was about four times that on sections with a change in curve of between 1 and 5 degrees;
   d. For changes in curve of greater than 15 degrees, the average accident rate was even higher.
3. For individual lane widths, the differences in accident rates between lanes of 3.7 m (12 ft) and 3.4 m (11 ft) were more or less more pronounced than those between lanes of 3.4 m (11 ft) and 3.1 m (10 ft).

Typical relationships between degree of curve and accident rates are shown in Figure 3 for the United States and Germany (western) (*49*).

Gradient

The operating speed of a vehicle is influenced by the characteristics of the vertical alignment. Trucks and buses suffer the most on grades, especially on upgrades, where a speed reduction may become significant [Rotach (*50*)]. On downgrades, trucks and buses are often driven at crawl speeds to maintain control for the effect of providing longer braking distances. On longer downgrade sections, with high longitudinal grades, brakes may not adequately slow down a heavy vehicle traveling at high speed and bring it to a stop. For passenger cars, longitudinal grades also lead to a variation in operating speeds, but not in a manner that is as pronounced as that for trucks. It may be concluded that, with increasing longitudinal grades, an increase in the nonhomogeneity of traffic flow could increase the risk of an accident.

A number of studies concerning the relationship between vertical alignment and accident risk have been done.

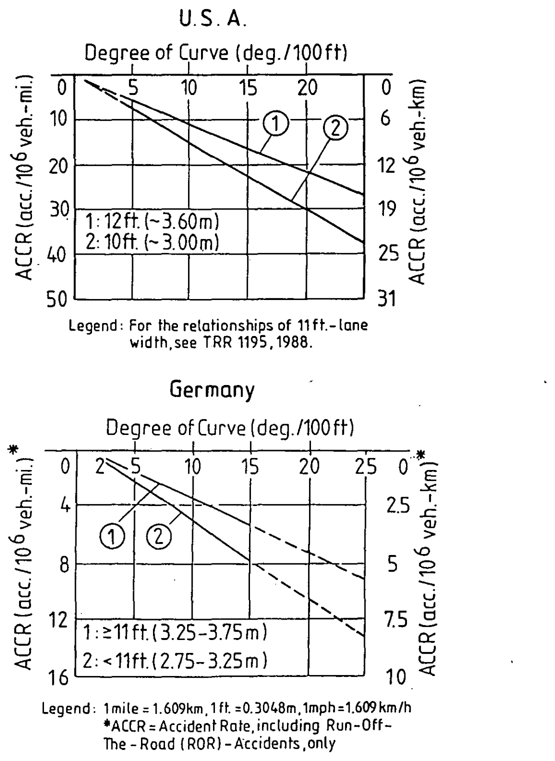


FIGURE 3 Nomogram for evaluating accident rates as related to degree of curve. TRR is *Transportation Research Record.*

An investigation by Bitzl, which is cited by Pucher (*51*), for German two-lane rural roads established a positive relationship between gradient and accident rate. In other words, the accident rate increases as the gradient increases. In another study related to German expressways, Bitzl (*52*) found a marked relationship between grade and accident rate. He indicated that steep grades of 6 to 8 percent produce over four times the number of accidents as gradients of less than 2 percent.

Vasilev (*41*) determined in the former USSR that accident rates were especially high on steep grades. In a study that evaluated data bases from Germany, Great Britain, and the former USSR, Silyanov, (*24*) indicated that the accident rate increased as the gradient increased. Similar results were reported by Babkov (*42*) in the former USSR.

Studies cited by Pignataro (*25*) did show that steeper grades increase the accident rates and skidding accidents on two-lane rural curved sections.

Krebs and Kloeckner (*8*) analyzed accident data for two-lane rural roads in the FRG. They indicated that the accident rate showed a slight increase up to grades of about 6 percent. For grades of more than 6 percent, a sharp increase in the accident rates was noted. Studies by the authors (*5–7*) indicated that grades of up to 5 percent did not have any particular effect on the accident rate.

For two-lane rural highways, the research studies reported above have generally shown that

1. Grades of less than 6 percent have relatively little effect on the accident rate.
2. A sharp increase in accident rate was noted on grades of greater than 6 percent.

Figure 4 illustrates the relationship between the accident rate and accident cost rate and the gradient (*31*) for new designs and redesigns made according to the German design guidelines (*9*). From Figure 4 it can be seen that

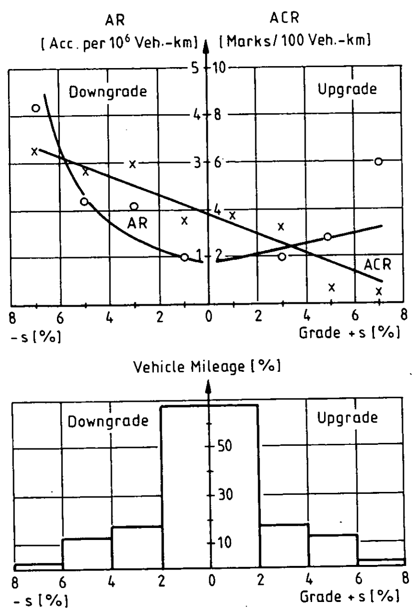


FIGURE 4 Accident rate and accident cost rate versus grade.

1. Longitudinal grades of between 0 and ±2 percent show the most favorable results. With increasing upgrades, the accident rate gradually increases, whereas with increasing downgrades, the risk of being involved in an accident increases exponentially.
2. Between upgrades of +7 percent and downgrades of −7 percent, the accident cost rate gradually increases; this is understandable, since operating speeds are highest on steep downgrades.

Sight Distance

Sight distance, which is dependent on both horizontal and vertical alignments, is of great importance to traffic safety. Hiersche (*53*), in the FRG, pointed out that sight distance is the most important criterion in the design of highway alignments. Krebs and Kloeckner (*8*) did not fully agree with that statement, but said that insufficient sight distances are the cause of many accidents. Meyer et al. (*54*) stated that about one-quarter of all rural accidents originate from overtaking maneuvers for which passing sight distances were not sufficient. Similar results were reported by Netzer (*55*) in the FRG, who determined that passing maneuvers accounted for about 21 percent of all traffic accidents.

An analysis of accidents on U.S. roads by Young (*56*) showed that the accident rate correlated negatively with sight distance. For a sight distance of less than 240 m (790 ft), the accident rate was twice as high as that for a sight distance of more than 750 m (2,450 ft).

In a German investigation, Bitzl and Stenzel (*57*) reported that the frequency of accidents related to improper passing maneuvers sharply increased for sight distances of less than 400 to 600 m (about 1,300 to 2,000 ft).

Sparks (*58*) established a negative relationship between stopping sight distance and accident rate in the United States. Similar results were reported by Silyanov (*24*) in the former USSR and Kunze (*26*) in the FRG.

Another study of accidents on two-lane rural roads in Germany by Krebs and Kloeckner (*8*) determined the following:

1. As sight distance increases, the accident risk decreases.
2. High accident rates were associated with sight distances of less than 100 m (330 ft).
3. With sight distances of between 100 m (330 ft) and 200 m (660 ft), accident rates were about 25 percent lower than those associated with sight distances of less than 100 m (330 ft).
4. For sight distances of more than 200 m (660 ft), no major improvements in accident rates were noted.

Studies by the authors (*5–7*) determined that sight distances of more than 150 m (490 ft) did not have any particular effect on accident rates.

A study of accidents on two-lane rural roads in Texas by Urbanik et al. (*59*) indicated that limited sight distances, especially on crest vertical curves, could cause a marked increase in accident rates. An example would be a sharp horizontal curve hidden by a crest vertical curve.

The research studies reported above have established a negative relationship between available sight distance and accident risk. However, it can be hypothesized that other influencing parameters, such as wide pavements and gentle radii of curve, might also play a part in the observed positive effect of greater sight distances on the accident situation. For narrow road sections, an increase in sight distances could favorably affect traffic safety.

Traffic Volume

When analyzing the relationship between AADT and accident rate, one must keep in mind that road sections with high traffic volumes normally have good designs (i.e., wide pavements, gentle curvilinear alignments, low gradients, etc.). This fact alone plays an important role in the investigation of the relationship between traffic volume and the accident situation.

Goldberg (*60*) investigated accidents on rural two-lane highways in France. For traffic volumes of up to 20,000 vehicles per day, he established a U-shaped distribution between accident rate and traffic volume.

Paisley, in a report by Wilson (*44*, p. 36), studied the effect of traffic volume on accident severity in Great Britain. For fatal accidents, he indicated that accident rate decreased as traffic volume increased. However for accidents with injuries that the accident rate increases as the traffic volume increases. Paisley's study was based on traffic volumes of up to 10,000 vehicles per day.

Roosmark and Fraeki (*61*) analyzed accident types on roads in Sweden with traffic volumes of up to 11,000 vehicles per day. They established the following results:

1. For single-vehicle accidents, the accident rate decreased as the traffic volume increased.
2. For multiple-vehicle accidents, the accident rate increased as the traffic volume increased.

An investigation of accidents on two-lane rural roads in Austria by Knoflacher (*62*) established a U-shaped distribution between accident rate and traffic volume. Accident rate was at a minimum for traffic volumes of between 6,000 and 6,500 vehicles per day. (For traffic volumes of less than 6,000 to 6,500 vehicles per day, single-vehicle accidents dominated. For traffic volumes of more than 6,000 to 6,500 vehicles per day, multiple-vehicle accidents prevailed).

For traffic volumes of up to 10,000 vehicles per day, Lamm and Kloeckner (*63*) in the FRG reported that the accident rate decreased as the traffic volume increased. They indicated that the level of design correlated highly with traffic volume, a result that could explain the favorable trends in accident rates on high-volume roads. Krebs and Kloeckner (*8*) found a negative linear relationship between accident rate and traffic volume of up to 16,000 vehicles per day in the FRG. Research studies by the authors (*5–7*) established a nonsignificant negative relationship between accident rate and traffic volume of up to 5,000 vehicles per day.

In conclusion, although some of the research studies reported above have shown that the accident rate decreases as the traffic volume increases, other investigations established a U-shaped distribution between accident rate and traffic volume. The U-shaped distribution was shown to be valid for multilane highways by Pfundt (*36*), Gwynn (*64*), and Leutzbach et al. (*65*). Leutzbach et al. (*66*) said that the U-shaped distribution between accident rate and traffic volume was also valid for two-lane rural highways.

Design Speed

In addition to the effect of pavement width, radius of curve/degree of curve, gradient, sight distance, and AADT on accident rate, Hiersche et al. (*31*) studied the effect of design speed on accident rate and accident cost rate. Table 1 gives an overview of their results for new alignments designed according to the German design guidelines (*9*). An examination reveals that the accident rate decreases as design speed increases from 60 to 80 km/hr (35 to 50 mph). For design speeds of greater than 80 km/hr (50 mph), the accident rate did not experience an improvement. The accident cost rate, on the other hand, increased as the design speed increased.

These findings are interesting, but further research, including large data bases, is clearly needed on this subject.

SUPERIMPOSITION OF DESIGN PARAMETERS AND DETERMINATION OF BREAKPOINT IN SAFETY

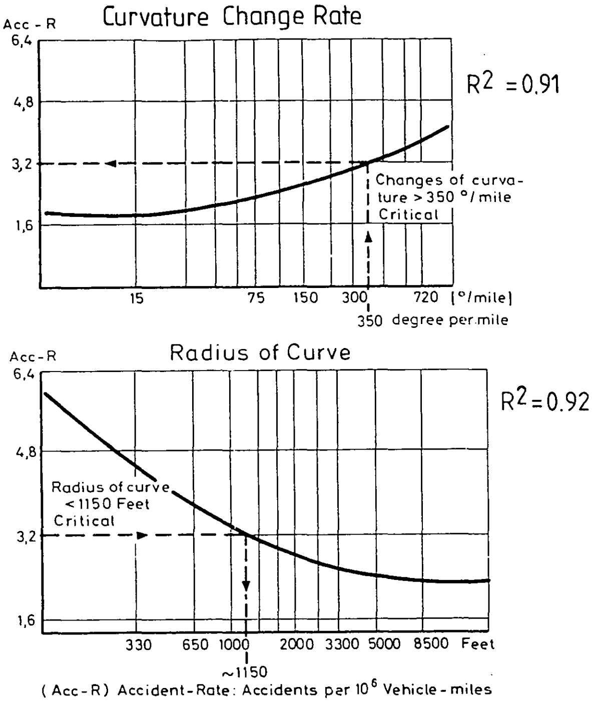
A number of researchers have used regression analysis to obtain quantitative estimates of the effects produced by design and traffic volume parameters on accident rates (*40, 67–74*). However, their findings did not provide any practical applications and did not yield any clue to the level of design parameters, such as degree of curve, above which improvements in traffic safety become particularly important.

Research by the authors (*5–7*), which analyzed the joint effects of several parameters—degree of curve, length of curve, superelevation rate, gradient of up to 5 percent, sight distance, lane width, shoulder width, and AADT—on operating speeds and accident rates, determined that the degree of curve explained most of the variation in the expected operating speeds and accident rates on curved sections of two-lane rural highways. On the basis of previous research (*5–7*), recommendations for good, fair, and poor designs were developed. Readers who are interested in detailed discussions of these recommendations should consult several previously published reports (*49, 75–79*).

Studies conducted at the Institute of Highway and Railroad Engineering at the University of Karlsruhe, Karlsruhe, FRG (*8,17,32*), yielded the results shown in Figures 5 to 8. The data

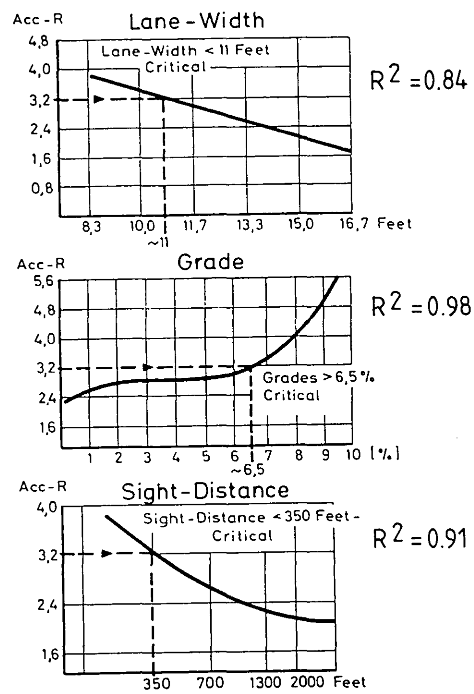
TABLE 1 Effect of Design Speed on Accident Rate and Accident Cost Rate

Design Speed (km/hr)	Vehicle Mileage (Absolute) (10^6 vehicle km)	Vehicle Mileage (%)	Accident Rate (no. of accidents/10^6 vehicle km)	Accident Cost Rate (DM/10^2 vehicle km)
60	52.3	14.6	2.12	3.62
70	18.0	5.2	1.78	3.85
80	234.5	65.7	1.15	4.21
100	52.0	14.5	1.11	5.21



Legend: 1mile = 1.609km, 1ft. = 0.3048m, 1mph = 1.609 km/h

FIGURE 5 Accident rate as function of curvature change rate and radius of curve on two-lane rural roads.



Legend: 1mile = 1.609km, 1ft. = 0.3048m, 1mph = 1.609 km/h .

FIGURE 6 Accident rate as a function of lane width, grade, and sight distance on two-lane roads.

base covered 4 years of accident experience (14,200 accidents) on 1162 km (722 mi) of two-lane roadways in western FRG. Figures 5 and 6 show that there is a definite correlation between accident rate and curvature change rate, radius of curve, lane width, gradient, and sight distance. However, the results in Figures 5 and 6 do not yield a clue to the level of design parameters above which improvements in traffic safety become particularly important. In other words, the results do not present a minimum level of safety or a maximum level of unsafety (*17*). [Curvature change rate is the absolute sum of the angular changes per section length of roadway with similar road characteristics. It has been used in the German design guidelines since 1973 (*9*).] For single curves, the curvature change rate formula reduces to just the degree of curve formula (*77–79*) (Table 2).

A number of studies conducted in Europe and the United States (*32,34,49,77,80*) have shown that curvature change rate correlates highly with operating speeds and accident rates. As shown in Figure 7, an increase in curvature change rate leads to a decrease in operating speeds. Furthermore, for new designs—designed according to the German design guidelines (*9*)—Lamm (*17*) indicated that curvature change rates greater than or equal to 250 gon/km (350 degrees/mi) produced relatively high accident rates. It should be noted that for new designs and redesigns in Germany, curvature change rates of greater than 250 gon/km (350 degrees/mi) are used in very few cases (Figure 7).

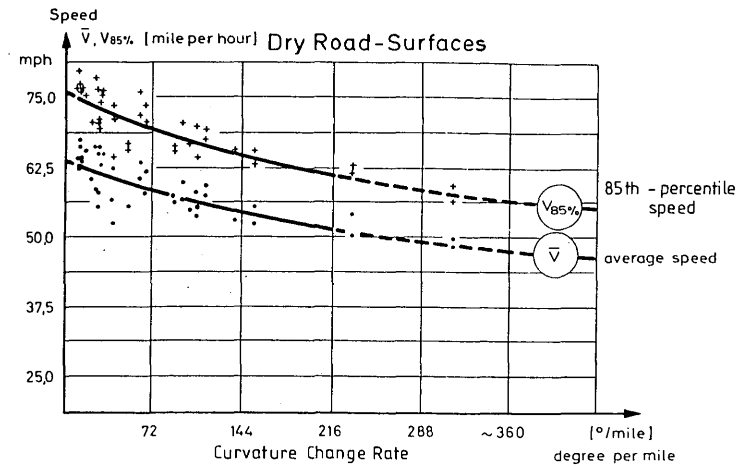
Lamm (*17*) concluded, "If curvature change rate of 350 degrees per mile is the highest acceptable level of curvature in modern geometric design guidelines, then the accident rate corresponding to this curvature change rate should be regarded as the break-point between levels of safety and un-safety." On the basis of the data in Figure 5 and Table 2 an accident rate of 2.0 accidents per 10^6 vehicle km (3.2 accidents per 10^6 vehicle mi) corresponds to a curvature change rate of 250 gon/km (350 degrees/mi) and a radius of curve of 350 m (1,150 ft) or a degree of curve of about 5 degrees. Furthermore, Lamm suggested that design parameters, such as radius of curve, lane width, and gradient, should be laid out during the design stages (for new designs or redesigns) in such a way that when the road is in full operation the accident rate should not be allowed to exceed 2.0 accidents per 10^6 vehicle km (3.2 accidents per 10^6 vehicle mi); this corresponds to a level of safety/unsafety of $1 - (2 \times 10^{-6}) = 0.999998$, or a 99.9998 percent chance that an accident will not occur.

Despite this high percentage, Lamm indicated that decisive endangerments could still be expected. To illustrate this point, he gave the following example:

> For a safety level of 100% . . . ADT of 10,000 vehicles per day . . ., it can be estimated that 3,650,000 (365 × 10,000) vehicles per year would pass a section-kilometer without being involved in accidents. However, according to the proposed level of safety/un-safety (99.9998%), only 3,649,993 vehicles would pass safely; that means, 7 vehicles would be involved in collisions on the observed section-kilometer. (*17*)

From Figures 5 and 6, an accident rate of 2.0 accidents per 10^6 vehicle km (3.2 accidents per 10^6 vehicle mi) corresponds to

- Radius of curve of about 350 m (1,150 ft),
- Lane width of about 3.3 m (11 ft),
- Grade of about 6.5 percent, and
- Sight distance of about 100 m (350 ft).



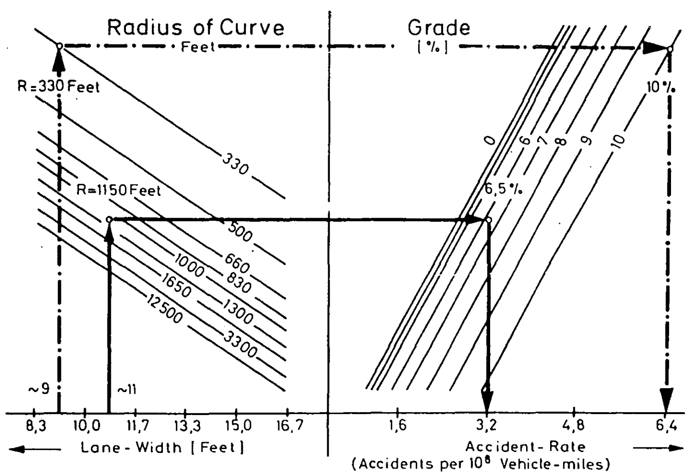
Legend: 1 mile = 1.609km, 1 ft. = 0.3048m, 1mph = 1.609 km/h

FIGURE 7 Relationship between operating speed and curvature change rate on two-lane rural roads.

Falling short of or exceeding the above values could result in an accident rate of greater than 2.0 accidents per 10^6 vehicle km (3.2 accidents per 10^6 vehicle mi). It should be noted that AASHTO's design guidelines appear to meet the minimum requirements when they recommend lane widths of 3.3 m (11 ft) and 3.6 m (12 ft), maximum grades of 6 to 7 percent—for design speeds of between 80 km/hr (50 mph) and 95 km/hr (60 mph) in mountainous topography, and a stopping sight distance of about 100 m (350 ft) for a design speed of 80 km/hr (50 mph). The German design guidelines show similar results.

For future designs, the use of radii of curve of less than 350 m (1,150 ft) should be carefully considered. For safety reasons, curvature change rates of greater than [350 degrees/mi (5 degrees of curve)], pavement widths of less than 3.3 m (11 ft), gradients of more than 6.5 percent, and sight distances of less than 100 m (350 ft) should be avoided. For responsible agencies, policy makers, and so on, the question here becomes, Which is more important —safety, economics, or the environment? This decision must be made on a case-by-case basis.

Up to now, the risk of an accident, as measured by the accident rate, was regarded as a function of single design parameters. Since the accident situation cannot be described by just one design parameter, as noted previously, Krebs and Kloeckner (*8*) studied the joint impacts of several design parameters, including radius of curve, lane width, gradient, sight distance, and traffic volume on accident rate. Sight distance was later removed from the analyses



Legend: 1 mile = 1.609km, 1 ft. = 0.3048m, 1mph = 1.609 km/h

FIGURE 8 Nomogram for determining accident rate as a function of lane width, radius of curve, and grade on two-lane roads.

TABLE 2 Relationship Between U.S. Design Parameter Degree of Curve and German Design Parameter Curvature Change Rate for Curves Without Transition Curves

"Degree of curve" DC	"Curvature Change Rate" CCR
• Definition $DC_{ft} = \frac{360°}{2\,\pi\,R} \left[\frac{Degree}{100\,ft}\right] = \frac{5729,6}{R} \left[\frac{Degree}{100\,ft}\right]$ DC_{ft} : radius of curve [ft]	• Definition (gon related to 400°) $CCR = \frac{\sum_{i=1}^{n} \lvert\gamma_i\rvert}{L} \left[\frac{gon}{km}\right]$ γ_i = change of angle (angular change) L = length of road section [km]
• in gon / 100 m $DC_m = \frac{6370}{R} \left[\frac{gon}{100\,m}\right]$ R = radius of curve [m]	• for one circular curve without transition curves $CCR = \frac{\frac{L_{cr}}{R} \times 63,7}{L} \left[\frac{gon}{km}\right]$ L_{cr} = length of circular curve [m] R = radius of curve [m]
• with gon / km $DC_m = \frac{63700}{R} \left[\frac{gon}{km}\right]$ R = radius of curve [m]	• with $L = L_{cr}/1000$ [km] $CCR = \frac{63700}{R} \left[\frac{gon}{Km}\right]$ R = radius of curve [m]

Relationship : $DC_{ft} \left[\frac{Degree}{100\,ft}\right] \times 36,5 \approx CCR \left[\frac{gon}{km}\right]$

36,5 [-] = conversion factor between DC_{ft} (Imperial System) into CCR (Metric System)

because it correlated highly with radius of curve. The same was true for curvature change rate. Traffic volume was also excluded because it did not affect the accident rate significantly. As a result of the study, Figure 8 was developed. Examination of Figure 8 leads to the following interesting results:

- Use of the design parameters recommended above results in an accident rate of 2.0 accidents per 10^6 vehicle km (3.2 accidents per 10^6 vehicle mi).
- Use of minimum design standards, such as German design guidelines, results in an accident rate twice as high as that recommended.

Once again, the proposed breakpoint in safety, as related to the accident rate, was obtained even when several design parameters were superimposed.

It should be noted that one unfavorable design parameter should not be superimposed on others. By correctly applying Figure 8, a high accident rate resulting from one unfavorable design parameter could be taken care of, at least partially, by proper selection of the other design parameters.

CONCLUSION

As a result of the present review, the following important conclusions can be drawn.

Influence of Pavement Width

- A distinct tendency for accident rates to decrease with increasing pavement width up to about 7.5 m (25 ft) was established.

Influence of Radius of Curve/Degree of Curve

- A negative relationship between radius of curve and accident rate was established.
- The same radius of curve in a sequence of similarly tuned radii can have effects on the accident situation other than those in a nontuned sequence of different radii, as is usually the case on most old alignments.
- For radii of less than 200 m (660 ft), the accident rate is at least twice as high as that on a radius of 400 m (1,300 ft). For radii of greater than about 400 to 500 m (1,650 ft), an increase in radius leads to a low-level safety gain.
- The most successful parameter in explaining the variability in accident rates was the degree of curve or curvature change rate. [On the basis of this parameter, recommendations for good, fair, and poor designs that were based on operating speeds and accident rates were made for the United States and Germany (*5–7,49,75–78*).]

Influence of Gradient

- Gradients of less than 6 percent have a low impact on the accident situation. For gradients of more than 6 percent, a sharp increase in the accident rate was noted.

Influence of Sight Distance

- A significant, positive relationship between sight distance and radius of curve was established.

Influence of AADT

- A negative relationship between traffic volume and road traffic accidents was established. Run-off-the-road accidents were found to decrease with increasing AADT up to 10,000 vehicles per day. Newer investigations report a U-shaped distribution between accident rate and traffic volume.

Superimposition of Design Parameters/BreakPoint in Safety

- Strong correlations between a number of design parameters and accident rates were established.
- A breakpoint in safety, or an accident rate of 2.0 accidents per 10^6 vehicle km (3.2 accidents per 10^6 vehicle mi), was proposed. Also proposed were limiting values for design parameters corresponding to this breakpoint in safety. Falling short of or exceeding those limiting values could mean that the breakpoint in safety is exceeded. This breakpoint in safety should be considered the borderline between levels of safety and unsafety.

It is not the intention of the authors to give the impression that the limiting values proposed in this paper are generally valid, since the study was mainly related to the influence of the road itself on traffic safety. As noted earlier, there are many factors that may exhibit a measurable influence on driving behavior and traffic safety on two-lane rural highways. These include, but are not limited to, human factors, physical features of sites, the presence and the action of traffic, legal issues, environmental factors, and vehicle deficiencies. All of these therefore constitute a complex mix of various causes of traffic accidents, of which the road itself represents only one factor, but a very important one.

Because accident cost rate, an indicator of accident severity, was shown in the present study to increase with increasing design speed and pavement width and decreasing grade, the authors propose that further studies with large data bases be conducted in this field.

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Horizontal Curve Design: An Exercise in Comfort and Appearance

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AASHTO's 1990 *A Policy on Geometric Design of Highways and Streets* contains information on procedures for three superelevation designs: rural highways and high-speed urban streets, low-speed urban streets, and curvature of turning roadways and curvature at intersections. The history of the horizontal curve design procedures through the published policies (1940 to 1990) is reviewed, the findings from the literature on key issues are presented, and additional research needs on side friction factors and transition length determination are discussed. The side friction factors used in high-speed and low-speed design were determined by using vehicle occupant comfort as the selection criterion. This criterion assumes that drivers limit their speed on curves to ensure comfort and that discomfort is directly related to the unbalanced side friction. Several concerns or issues accompany these assumptions. For example, the speed at which discomfort (or side pitch) first becomes noticeable may be slower than necessary for comfort or safety, and the level of discomfort felt by a driver may not be solely related to side friction only. These assumptions also do not directly consider vehicle characteristics or constant safety factors over the range of design speeds. Transition length determination for high-speed and intersection design is based on appearance and comfort. The criterion was developed to avoid an appearance that results from too rapid a change in superelevation. For low-speed design, a change in acceleration over the change in time factor, known as *C*, is used to determine superelevation runoff. High-speed design includes factors that are to be used to determine runoff lengths for roads with more than two lanes. Low-speed design does not include similar factors that adjust for wider pavements; however, it does include a method for adjusting runoff length for radii larger than the minimum that the high-speed design procedure does not include. Three research areas were identified on the basis of the present findings: (a) selection of side friction factors, (b) determination of transition lengths, and (c) evaluation of the need for and basis of the three different design procedures (high speed, low speed, and curvature at intersections). Research is needed in these areas because current practice is largely based on limited empirical data and existing practice without supporting material. Efforts to address these issues would require substantial funds.

Highway geometric design and safety issues are a constant challenge. Many of the concerns facing the industry today also were a problem in the 1920s and 1930s. In the early part of the century the existing system needed to be reconstructed to accommodate the needs of motorized vehicles rather than horse-drawn traffic. The surfaces needed to be stronger and the alignment redesigned to accommodate higher operating speed. During these early road-building days, procedures for horizontal curve and superelevation design were developed. In the design of roadway curves, it is necessary to establish the proper relationship of speed and curvature with superelevation and side friction. Although these relations stem from the laws of mechanics, the actual values selected for the design depended on practical limits and factors determined more or less empirically over the range of variables involved.

AASHTO's 1990 *A Policy on Geometric Design of Highways and Streets* (*1*) (commonly called the Green Book) includes information on three superelevation design procedures:

- Rural highways and high-speed urban streets,
- Low-speed urban streets, and
- Curvature of turning roadways and curvature at intersections.

These procedures are referred to in this paper as high-speed, low-speed, or intersection design, respectively. The high-speed design is for use on all rural highways, on urban freeways, and on urban streets where speed is relatively high and relatively uniform. Low-speed design is used for through roads and streets in urban areas where the use of superelevation is impractical and where drivers have developed a higher threshold of discomfort. Intersection design is used for curvatures of turning roadways and curvatures at high-speed, at-grade intersections.

The objective of the study (*2*) that formed the basis of this paper was to identify the research needs in horizontal curve design by using information from a historical review and a literature search on horizontal curve design. The historical review identified how the current procedures were developed and how the procedures have evolved over the past 80 years. This review was conducted primarily by reviewing seven different design policies (*1,3–8*) and early textbooks. The literature review provided information on the issues examined and also assisted in identifying and clarifying the issues needing additional research.

OVERVIEW OF SUPERELEVATION

When a vehicle moves in a circular path, it is forced radially outward by centrifugal force. Superelevation is the rotating of the roadway cross section to offset the centrifugal force acting on a vehicle traversing a curved section. For each combination of curve radius and travel speed, there is a specific superelevation that will precisely balance the centrifugal force. When a vehicle travels at speeds greater than those at which the superelevation balances all of the centrifugal force, side friction is needed to keep the vehicle on the curved path.

Point-Mass Equation

In the design of highway curves, a mathematical relationship exists among design speed, curvature, superelevation, and side friction. When a vehicle moves in a circular path, it is forced radially

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outward by centrifugal force. The centrifugal force is counterbalanced by the vehicle weight component related to the roadway superelevation or the side friction developed between the tires and the surface or by a combination of the two. By using the laws of mechanics, the basic point-mass (curve) formula derived to represent vehicle operation on a curve is:

$$e + f = \frac{V^2}{127R} \tag{1}$$

where:

e = rate of roadway superelevation (m/m),
f = side friction factor,
V = vehicle speed (km/hr), and
R = radius of curve (m).

The above equation is used to determine the minimum radius of a curve for a specific superelevation rate and side friction factor. On the basis of accumulated research and experience, the Green Book presents limiting values for superelevation and friction. These values vary in the different design categories included in the 1990 Green Book (high speed, low speed, and intersection design).

Rates

If a radius selected for a curve is greater than the minimum radius determined from Equation 1, then the designer uses a superelevation rate that is less than the maximum superelevation assumed. Tables, figures, or both, are included in the Green Book for this purpose. These tables and figures were developed on the basis of an assumed distribution of superelevation rates and side friction factors. Several methods are available for distributing superelevation and friction over a range of curves.

The Green Book lists five methods: Method 1, straight-line relation; Method 2, counteracting the centrifugal force with friction up to the maximum friction and then using a straight-line relation, increasing superelevation as the curvature increases up to maximum superelevation; Method 3, counteracting the centrifugal force with superelevation only until maximum superelevation is reached and then using a straight-line relation, increasing friction as the curvature increases up to maximum friction; Method 4, same as previous method, except that the method is based on average running speed instead of design speed; and Method 5, a curvilinear relation between superelevation and side friction.

The curvilinear relation (Method 5) is assumed for high-speed design. Low-speed design has the centrifugal force counteracted with friction until maximum friction is reached and then uses superelevation (Method 2). Method 2 was selected because "drivers [in urban areas] are more tolerant of discomfort, thus permitting employment of an increased amount of side friction for use in design of horizontal curves."

Side Friction

The side friction factor represents the friction present between the tires and the surface that is counteracting the unbalanced lateral force on a vehicle negotiating a curve. The upper limit of this factor is the point at which the tire is skidding or the point of impending skid. Because, as the Green Book states, "highway curves are designed to avoid skidding conditions with a margin of safety, the friction values should be substantially less than the coefficient of friction of impending skid." The Green Book also states "the portion of the side friction factors that can be used with comfort and safety by the vast majority of drivers should be the maximum allowable value for design." The values present in the 1990 Green Book for high-speed design are at "the point at which the centrifugal force is sufficient to cause the driver to experience a feeling of discomfort and cause him to react instinctively to avoid higher speed." Figure 1 compares the different friction factors for the three design methods.

Transition

Transition consists of superelevation runoff and tangent runout. The 1990 Green Book defines *superelevation runoff* as the general term denoting the length of highway needed to accomplish the change in cross slope from a section with adverse crown removed to a fully superelevated section, or vice versa. *Tangent runout* is the general term denoting the length of highway needed to accomplish the change in cross slope from a normal crown section to a section with the adverse crown removed, or vice versa. Table 1 compares the superelevation runoff lengths determined for each design procedure by assuming a 64.4-km/hr (40-mph) design speed.

DESIGN PROCEDURES

Each of the three design procedures included in the Green Book has a unique history. Procedures for high-speed and intersection design were included in AASHO policies published in the 1940s [the 1945 *Design Standards (Geometric) for Highways (Primary)* (*5*) for high-speed and the 1940 *A Policy on Intersections at Grade* (*3*) for intersection design]. The low-speed procedures were introduced in the 1984 AASHTO policy (*8*). The following are summaries of the superelevation rates, friction, and transition design histories for each of the three design procedures.

High-Speed Design

Superelevation Rates

Superelevation rates of as high as 0.08 m/m (ft/ft) were used during the 1920s. The 1941 AASHO policy (*4*) stated that the maximum rate is 0.12 m/m (ft/ft), but if snow and ice conditions prevail, the 0.08-m/m (ft/ft) rate should be used. These recommended rates in high-speed design are also present in the current policy. The method for distributing superelevation rates over radii larger than the minimum radii have not changed since they were first introduced in 1954.

Friction

The side friction factors present in the 1990 Green Book were determined from an assumed straight-line relation of data points from several studies conducted in the 1930s and 1940s. One of

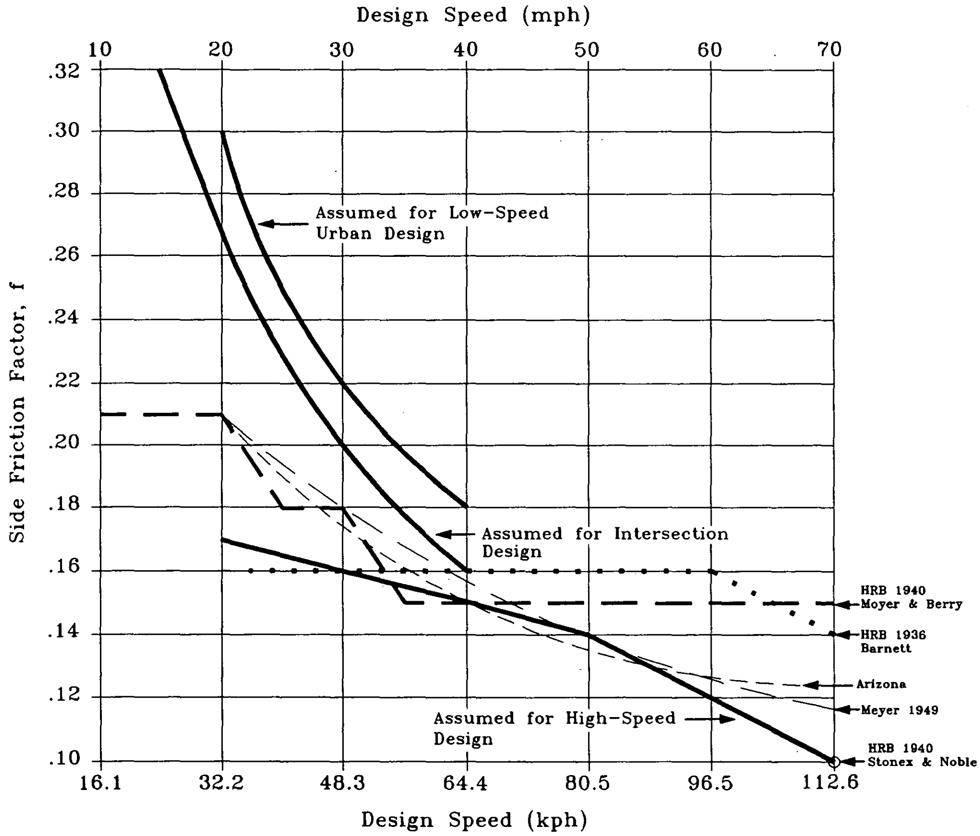

FIGURE 1 Comparison of friction factors.

TABLE 1 Comparison of Transition Designs

General assumptions: 64.4 kph (40 mph) design speed and 2-lane roadway with 3.7-m (12-ft) lanes		
High Speed Design	**Low Speed Design**	**Intersection Design**
Assumption Superelevation rate = 0.06	Assumption • Superelevation rate = 0.06	
Calculated/Determined • Friction factor = 0.15 (1990 Green Book Table III-6) • Minimum radius = 155.3 m (509 ft) (1990 Green Book Table III-6) • **Runoff length = 38.1 m (125 ft)** (1990 Green Book Table III-15)	Calculated/Determined • Friction factor = 0.178 (1990 Green Book Table III-6) • Minimum radius = 137.3 m (450 ft) (1990 Green Book Table III-6) • **Runoff length = 35.1 m (115 ft)** (1990 Green Book Table III-6)	Calculated/Determined • Friction factor = 0.16 (1990 Green Book Table III-17) • Minimum superelevation = 0.09 (1990 Green Book Table III-17) • Minimum radius = 131.1 m (430 ft) (1990 Green Book Table III-17) • Suggested **minimum length of spiral** (argued as what should be used as the transition length in previous editions of AASHTO Policies) = **48.8 m (160 ft)** (1990 Green Book Table III-18) • Maximum rate of change = 0.58 (1990 Green Book Table IX-13) calculation: **Runoff length** = 0.58 * 100 * 7.32 m * 0.06 = **25.5 m (84 ft)** • Note: the runoff length calculation is similar to the method used in high speed design except no discussion is included on a minimum runoff length.
Potential Changes • If the pavement is wider than two lanes, then use a 1.2, 1.5, or 2.0 conversion factor for a three-lane pavement, a four-lane undivided pavement, or a six-lane undivided pavement, respectively, to calculate the runoff length (1990 Green Book pages 178-179).	Potential Changes • If the radius used in the design is larger than the minimum radius of 137.3 m (450 ft), then the runoff length can be adjusted using information provided in the 1990 Green Book Figure III-20.	

those studies asked observers to report when they felt a "side pitch outward" when traversing a curve; another used a ball bank indicator and assumed that the 10-degree reading was the "value at which the driver of a car senses some discomfort and where the hazard of skidding off the curve becomes apparent." The factors based on those studies, which were not very different from the values included in the 1945 AASHO policy, were included in the 1954 AASHO policy. Only slight modifications of the friction values have occurred since then.

Transition Design

The 1-in-200 rate of cross slope change that is currently used to calculate superelevation runoff length [at the 80.4-km/hr (50 mph) design speed] was included in the 1941 AASHO policy. This rate is based on appearance; it determines a runoff length that is sufficient to avoid distorted appearance as the driver approaches a curve. Although the 1941 AASHO policy used the 1-in-200 rate for all design speeds, the 1954 AASHO policy used it for the 80.4-km/hr (50 mph) design speed and varied the rate for other design speeds. The 1954 AASHO policy also introduced a minimum runoff length that approximated the distance traveled in 2 sec at the design speed and factors for use in determining superelevation runoff lengths for roads with more than two lanes. The 1984 AASHTO policy included a discussion on determining the tangent runout.

Intersection Design

Superelevation Rates

The maximum superelevation rates listed in the AASHTO policies have not changed significantly in the past 50 or more years. In 1940, 0.10 m/m (ft/ft) was recommended for turning speeds of 64.4 and 80.4 km/hr (40 and 50 mph), and 0.05 m/m (ft/ft) "appears to be reasonable" for a turning speed of 48.3 km/hr (30 mph). The 1954, 1965, 1984, and 1990 policies contain similar material; the general range of maximum superelevation rates for curves is 0.06 to 0.12. The 1954 to 1990 policies include tables that list suggested superelevation rates in relation to design speed and radius of curve. These rates were "derived in much the same manner as for open highway curves."

Friction

The 1940 AASHO policy listed safety factors [1.3 at 32.2 km/hr (20 mph) to 1.6 at 80.4 km/hr (50 mph)] and coefficients of friction at impending skid. These values were multiplied to arrive at the design side friction factor used to determine minimum safe radii. The 1954 policy contained different friction factors than the 1940 policy and did not include a safety factor. The side friction factors were based on studies conducted to determine the distribution of speeds on intersection curves. A curve that "gives an average or representative curve" of the data and that used high-speed factors for one boundary and 0.5 for the other was drawn. Good (*9*), in his review of superelevation, commented that the plotted points represented averages of large vehicle samples, the scatter in the original data "would produce a diagram which defied the drawing of any trend line," and the apparent downward trend in the data depends rather critically on one or two data points. He also recommended [along with Harwood and Mason (*10*)] that a minimum radius is the result of a maximum assumed side friction factor and a maximum rather than a minimum superelevation rate. No changes to the information in the 1954 policy were made in 1965, 1984, or 1990 except for the addition of information on a 16.1-km/hr (10-mph) design speed in the 1984 and 1990 policies.

Transition Design

In the 1990 and 1984 AASHTO policies, superelevation runoff was calculated by using a "change in relative rate between the edge of a two-lane pavement and the centerline (in percent)." Earlier policies either used an equation commonly used to calculate a spiral (1940, 1954, and 1965 AASHO policies) or used a rate of cross slope change per 30.5 m (100 ft) of length (1954 and 1965 AASHO policies).

Low-Speed Urban Street Design

Procedures for low-speed urban street design were first introduced in the 1984 AASHTO policy. The reasons for the introduction of this new procedure in superelevation design were not included in the 1984 Green Book.

Superelevation Rates

The maximum superelevation rate listed in the 1984 and 1990 AASHTO policies is 0.04 or 0.06. The distribution of superelevation with curvature follows the assumption that the centrifugal force is counteracted in direct proportion by side friction up to the maximum assumed friction; then, superelevation is used in direct proportion until it reaches maximum superelevation.

Friction

The assumed friction curve (Figure 1) for low-speed urban design is "based on a tolerable degree of discomfort and provides a reasonable margin of safety against skidding under normal driving conditions in the urban environment." Explanations as to why different friction factors for low-speed versus high-speed or intersection design for a particular design speed exist, other than the above statement, are not provided [e.g., at 64.4 km/hr (40 mph), high-speed side friction is 0.15, intersection side friction is 0.16, and low-speed side friction is 0.178].

Transition Design

Superelevation runoff length is calculated by using an equation that includes a rate of change of the side friction factor called *C*. The *C* values are similar to the values used in the spiral length calculations in other sections of the policy; however, the source of the formula was not discussed in the policy. Detailed guidance on adjusting the lengths of superelevation runoff for radii that are larger than the minimum is provided.

RECENT RESEARCH

Several research studies have examined key components or issues of horizontal curve design. Following are summaries of the findings from research on friction factors and the point-mass equation. Another area of concern is how existing design practices affect trucks. A summary of findings from two research efforts on trucks is also included below.

Friction

Emmerson (*11*) in 1969 used car speeds on curves to calculate side friction factors. Approximately 80 percent of the vehicles experienced a side friction factor of less than 0.15 on curves with radii of between 351 m (1,150 ft) and 196 m (642 ft). Sites with very small radii [101 m (330 ft) and 21 m (70 ft)] had mean factors of 0.22 and 0.27, respectively. Glennon (*12*) in 1969 commented that the use of friction demand design values that correspond to that point at which side forces cause driver discomfort has no objective factor of safety relationship to the side friction capability of the tire-pavement interface.

Glennon and Weaver (*13*) conducted a study that examined vehicle paths, lateral skid resistance, and the need for safety margins. They recorded free-flowing vehicles on five horizontal curves to relate actual vehicle paths to the highway curve radius. Their data indicated that most vehicles experience their critical path maneuver near the beginning or end of the curve. Bell's (*14*) United Kingdom study in 1980 found results similar to those of Glennon and Weaver (*13*).

McLean (*15*) in 1983 argued that the side friction factor is a result of driver behavior rather than an explanation for it. His two major objections were that (a) there is no empirical evidence that drivers respond to actual or subjectively predicted side friction in the selection of curve speed rather than to some other parameter, and (b) owing to the interrelationship among speed, curve geometry, and side friction, attempts to represent driver behavior as a side friction-speed relationship may cloud the more fundamental issue of driver speed behavior and road conditions.

Lamm et al. (*16*), using regression models, compared side friction demand (determined on the basis of the radius and the superelevation present at the site and the 85th percentile speed) and the available or assumed side friction (determined on the basis of the procedures presented in the Green Book) for a range of degree of curve, operating speed, and accident rate values. They found that side friction demand exceeded the side friction assumed in the following situations: degree of curves greater than 6.5 degrees, curves with operating speeds of less than 80.4 km/hr (50 mph), and curves with accident rates of greater than 9.7 to 11.3 accidents per 10^6 vehicle km (6 to 7 accidents per 10^6 vehicle mi).

Point-Mass Equation

A 1980s FHWA study (*17,18*) found that the minimum level of tire-road friction identified for maintaining the stability of passenger cars was found to be equal to the "point-mass" design value for the curve. The minimum level of friction necessary for maintaining the stability of the five-axle tractor-semitrailer, however, was approximately 10 percent higher than the point-mass design value. The authors also concluded that no substantive evidence regarding friction factor dispersion could be identified to conclude that current highway curve design practice, on the basis of a point-mass formulation, should be modified to accommodate the observed wheel-to-wheel variations.

Truck Concerns

Harwood and Mason (*10*) and Harwood et al. (*19*) determined the margin of safety against skidding or rollover for a passenger car or truck on a horizontal curve and the speed at which skidding or a rollover would occur. They concluded that on lower-design-speed horizontal curves designed by using the high-speed design criteria, the most unstable trucks can roll over when traveling at as little as 8.0 to 16.1 km/hr (5 to 10 mph) over the design speed. This is a particular concern, they noted, on freeway ramps, many of which have unrealistically low design speeds in comparison with the design speed of the mainline roadway. In their analysis of superelevation design at intersections, Harwood and Mason (*10*) found that for design speeds of 16.1 and 32.2 km/hr (10 and 20 mph), a truck could skid or roll over by exceeding the design speed of a minimum-radius curve by 8.0 km/hr (5 mph) or less.

SUMMARY

The side friction factors that are currently used in the high-speed and low-speed design procedures were determined by using vehicle occupant comfort as the selection criterion. This criterion assumes that drivers limit their speed on curves to ensure comfort for the occupants of the vehicles, and discomfort is directly related to the unbalanced side friction. Several concerns or issues accompany these assumptions. For example, the speed at which discomfort (or side pitch) first becomes noticeable may be slower than necessary for comfort or safety, and the level of discomfort felt by a driver may not be solely related to side friction only. The above assumptions also do not directly consider vehicle characteristics or constant safety factors over the range of design speeds. Side friction factors for intersection design were based on studies conducted in the 1950s that determined the distribution of speeds on intersection curves.

The transition distance from a normal crown section to the superelevated curve for high-speed and intersection design is based on appearance and comfort. The criterion was developed to avoid an appearance that results from too rapid a change in superelevation. For low-speed urban street design, a change in acceleration over the change in time factor, known as *C*, is used to determine superelevation runoff. This *C*-factor is similar to the factor used to determine spiral lengths. High-speed design includes factors that are to be used to determine runoff lengths for roads with more than two lanes. Low-speed design does not include similar factors that adjust for wider pavements; however, it does include a method for adjusting runoff length for radii larger than the minimum that the high-speed design procedure does not include.

PROPOSED RESEARCH

Three research areas were identified on the basis of the findings of the present study. These areas are the selection of side friction factors, determination of transition lengths, and evaluation of the need for and basis of the three different design procedures (high speed, low speed, and curvature at intersections). Research is needed in these areas because current practice is largely based on limited empirical data and existing practice without supporting material. Different design criteria could also provide additional flexibility to a designer attempting to meet existing driveways or culverts when redesigning a roadway. The reader should note that efforts to address the following issues would require substantial funds and efforts.

Side Friction Factors Used in Superelevation Design

Problem

The side friction factors that are currently used in the high-speed and low-speed design procedures were determined by using vehicle occupant comfort (in the 1930s and 1940s) as the selection criterion. This criterion assumes that drivers limit their speed on curves to ensure comfort for the occupants of the vehicles, and discomfort is directly related to the unbalanced side friction. Several concerns or issues accompany these assumptions. For example, the speed at which discomfort (or side pitch) first becomes noticeable may be slower than necessary for comfort or safety, and the level of discomfort felt by a driver may not be solely related to side friction only. The above assumptions also do not directly consider vehicle characteristics or constant safety factors over the range of design speeds. Other issues that need to be investigated include whether vehicles in different lanes of a multilane roadway experience significantly different side friction forces, whether constant margin of safety values are needed, and if so whether these values should be based on trucks or passenger cars. The likelihood that vehicles will slide down an iced superelevated section when driving slowly or stopped and the combination of stopping friction needs and available side friction on a maximum-degree curve are other concerns expressed by designers.

Proposed Research

Evaluate the appropriateness of using comfort for a passenger car occupant in the selection of side friction factors. Identify and evaluate other potential criteria that could be used in selecting the side friction factors.

Different Design Procedures for Horizontal Curve Design

Problem

Currently, the Green Book includes three methods that can be used to design the superelevation of a horizontal curve: rural highways and high-speed urban streets, low-speed urban streets, and curvature of turning roadways and curvature at intersections. Are three different procedures justifiable? What should form the basis of each design procedure?

Proposed Research

The research should critically evaluate the existing horizontal curve design procedures (e.g., high speed, low speed, and curves at intersections) as well as investigate other potential procedures for designing a horizontal curve. It should also critically evaluate the basis of each design procedure. The research should conclude with a recommendation on what design procedures should be included in the AASHTO Green Book.

Transition Design

Problem

The transition distance from a normal crown section to the superelevated curve for open highway or high-speed design and for curves at intersections is based on appearance and comfort. The criterion was developed to avoid an appearance that resulted from too rapid a change in superelevation. For low-speed urban street design, a change in acceleration over the change in time factor, known as C, is used to determine superelevation runoff. This C-factor is similar to the factor used to determine spiral lengths.

High-speed design includes factors that are used to determine runoff lengths for roads with more than two lanes. Low-speed design does not include similar factors that adjust for wider pavements; however, it does include a method for adjusting runoff length for radii larger than the minimum that the high-speed design procedure does not include.

The use of runoff lengths that are shorter than the lengths provided in the Green Book could assist engineers in designing horizontal curves in developed areas where meeting existing cross-road grades is vital or in areas where the cost to purchase rights-of-way are high. Identification of the consequences of providing superelevation runoffs that are less than the values indicated in the 1990 Green Book is critical in making or supporting these design decisions.

Proposed Research

The research should critically evaluate current transition design for all three design procedures (high speed, low speed, and curvature at intersections) and propose and justify new transition lengths (or procedures to determine transition length). When transition lengths should or can be adjusted and by how much should also to be investigated and reported.

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Environmental Impact on Highway Geometric Design in Western Europe Based on a Geographical Information System

RUEDIGER LAMM, ARTUR K. GUENTHER, AND BERNHARD GRUNWALD

Attempts were made to reveal the interrelationships between environmental impacts and other important planning goals as well as between the design flow in modern highway geometric design guidelines, with special emphasis placed on environmental protection issues. The governments of most western European countries have enacted laws requiring Environmental Compatibility Examinations for traffic projects and also require engineers to conduct environmental compatibility studies (ECSs) that assess environmental impacts when planning highways. These studies must always be done before starting the design and construction phases of a highway construction project and must examine all existing natural, ecological, and cultural resources that are important to the integrity of an observed region. The ECS consists of two parts: (a) a space-related sensitivity investigation and (b) a comparison of alternatives. Geographical information systems (GISs) are recommended for use in conducting those studies. A case study involving a sensitivity investigation with thematic maps based on digitized geographical information is described. The thematic maps were developed to portray important environmental resources. By using a computer-supported GIS examination procedure, the thematic maps were superimposed, weighted, and evaluated to develop a new map. The new map, which incorporates those environmental resources, is used to identify the low-conflict corridor (the corridor that minimizes environmental effects) in the investigated region. Incorporation of the new design component, an ECS, into a computerized GIS format is a significant step forward in helping to present such work to the public for clearer understanding and general approval. It is recommended that future editions of the Green Book include material on ECSs.

"It is no question, that modern highway engineering has to respond to the mobility requirements of the citizen as well as to the highly developed economy. Providing traffic service to users is imperative and is certainly part of the quality of life, we are used to, today" (*1*). Alternately, the increasing impairment of the human living space concerns all people.

Therefore, the preservation and conservation of natural resources are important tasks for the future. The goals of protecting nature and the landscape as well as the soil and water and avoiding the creation of disturbances to the ecological balance have become priorities. However, *environmental protection* in western Europe is often used as an argument to abandon further completion of the highway network. In this connection it is recognized that (*1,2*)

1. The European governments have serious intentions toward nature and the environment. By abandoning the future completion of traffic routes, however, problems cannot be solved, economic growth and employment cannot be ensured, and traffic safety cannot be improved.
2. Environmental protection is no longer an empty formula for the highway engineer. At all planning, design, or construction phases, the relevant consequences of a project on the environment must be investigated, evaluated, and balanced against other public and private interests.
3. It must be understood, however, that by examining the environmental compatibility of a highway project, all planning stages and the resulting necessary adjustments become more difficult, more extensive, more time-consuming, and more expensive.
4. Thus, environmental protection is expensive. In most cases in western Europe it can be estimated that between 5 and 20 percent of overall project costs must be spent on environmental protection.

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BASIC PROCEDURE IN ROAD PLANNING AND DESIGN WITH SPECIAL EMPHASIS ON ENVIRONMENTAL PROTECTION ISSUES

The elaboration of a roadway design includes several design levels. With each level the designs become more concrete and the maps more detailed. At the same time, however, the remaining planning levels are always concentrated on fewer alternatives, since useless alternatives are excluded by such a selection procedure. In principle planning, design, construction, and operation must be regarded, meaning that the project must be harmless to the environment, safe, economical, and if necessary, efficient for traffic, and afterward the continued operation must be ensured (*3*).

Parallel to the technical elaboration, legal procedures take their course, by which harmonization of development planning, acquisition of land and soil, achievement of rights (water and trespassing rights, etc.), the extent of compensation, and the allocation of structural performances must be cleared and settled in accordance with public and civil laws and the design progress (Figure 1) (*4*).

The numerous relationships among driving behavior, road design, traffic flow, and environment require an iterative procedure for the establishment of the design. This means that first assumptions must be made, and during subsequent design stages it must be determined whether these assumptions are correct or incorrect (in the latter case, the design process must be repeated with better assumptions). Furthermore, a conclusive solution is almost impossible; this means that for present problems, different alternatives (which cover all of

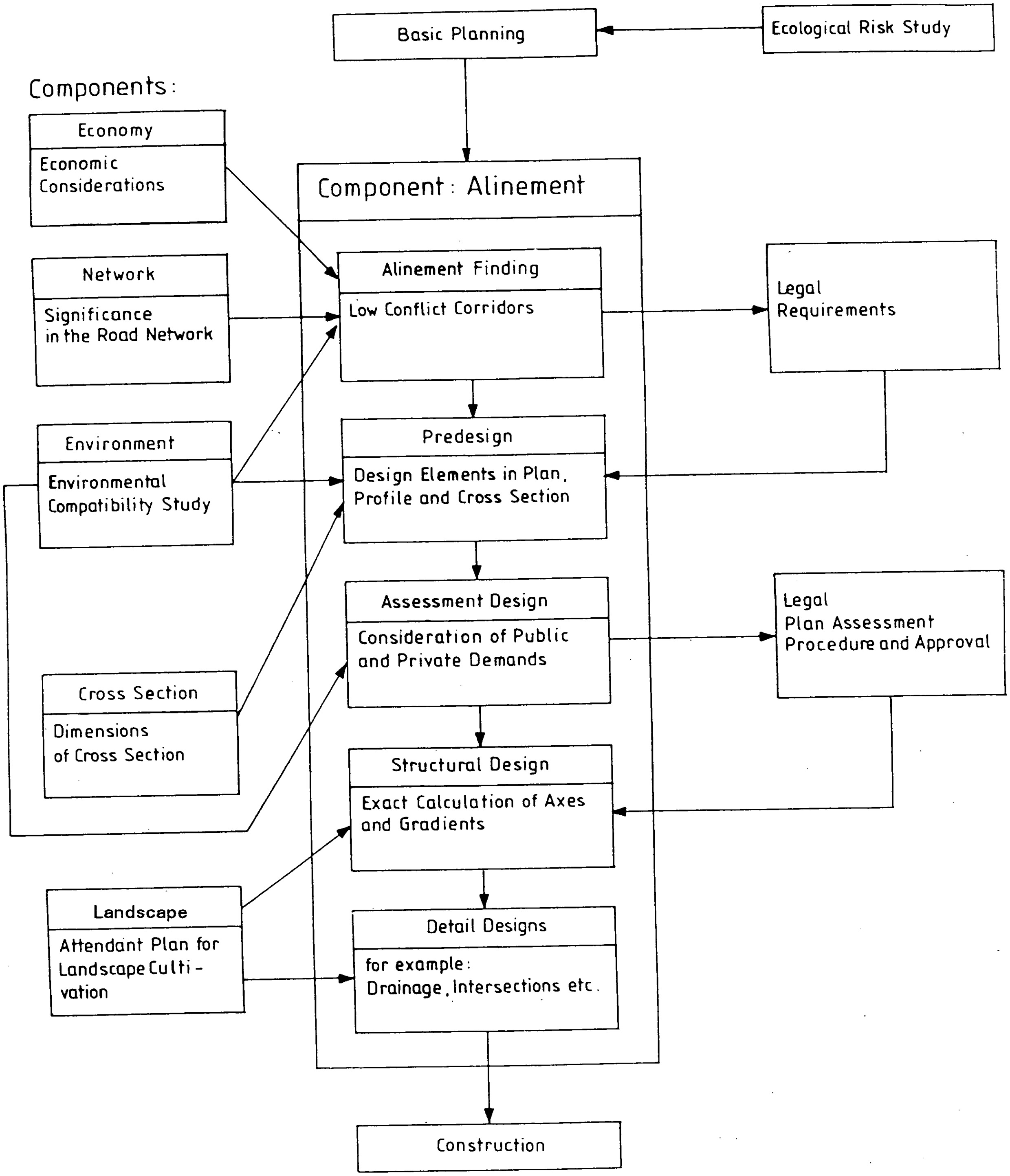

FIGURE 1 Design levels in the range of validity of modern highway geometric design guidelines with special emphasis on environmental protection issues.

the possibilities as much as possible) must be investigated so that designers can select the most appropriate one.

In planning federal and state routes the responsible authorities are in general required to participate at all planning levels. The basis for this in Europe, and especially in the Federal Republic of Germany, is the "Law for the Realization of the Guidelines of the Board of June 27, 1985 for the Environmental Compatibility Examination (ECE) for Specific Public and Private Projects (85/337/EWG)" (5) of February 12, 1990.

According to this law, uniform principles for effective environmental precautions must be applied; this means that the effects on the environment must be described and evaluated, and the result of the ECE must be considered in all official decisions related to the approval of a traffic project.

Thus, for the construction or alteration of a federal or state route, which must undergo a legal plan assessment (development plan), the law requires that an ECE be performed (*5*). The ECE procedure is regulated by law; it is a continual and integrated part of the levels of planning for alinement determination and plan assessment.

The ECE is based on an environmental compatibility study (ECS). The *Instructural Guide for the Environmental Compatibility Study in Highway Planning (ECS)* is used (*6*). The ECS must normally be conducted in two elaboration steps (Figure 2).

1. A space-related sensitivity investigation, which includes goal-oriented space analysis and goal evaluation, establishment of relatively low conflict corridors for the alinement, and allocation of specific conflict areas.
2. Comparison of alternatives, which includes a comparative evaluation of alternatives, including the rehabilitation or restoration alternative and the zero (do nothing) alternative.

During the first level of planning, during which alternative routes are determined, graphical elaboration is sufficient (Figure 1).

In further evaluations, the following route determination criteria should be considered:

- Traffic-related goals of regional planning;
- Environment-related evaluation criteria, including actual use and biotype function, present soil and water function, landscape quality, dwelling function, recreational function, cultural and other resources worthy of protection, and climate and air;
- Soil conditions;
- Vicinities of settled areas;
- Section length, sizes of radii of curves, and gradient;
- Necessary engineering structures;
- Need for demolition of buildings; and
- Construction time and costs.

The result of the alinement finding is the selection of a priority alternative, which is based on the comparison of alternatives (Figure 2).

The next planning phase is predesign (Figure 1), in which the selected routes for one or a few different alternatives are examined and the design elements for cross section and alinement are determined. Axes and gradients are calculated at least for the main points and the constraint points. More exact information about impairment of the environment and residents, construction and operation costs, and road user costs is available at this phase. In many cases legal procedures can still take place at the predesign phase. However, today, a more profound at least partial, elaboration is often necessary.

After the predesign is approved, the structural design, which represents further development of the predesign, is made. All axes, gradients, drainage, and the exact need for property can be determined by performing calculations. The structural design serves as the basis for the acquisition of land, the invitation of tenders, and the allocation of structural performance (Figure 1).

For single parts, for example, drainage and intersections-interchanges, or for the whole construction process, further detailed designs may still become necessary.

At the level of these detailed designs, interferences with nature and the landscape must be presented separately according to the required compensation and replacement procedures discussed in the attendant plan for landscape cultivation (*7*).

In Figure 1 the range of validities suggested for geometric design guidelines for modern highways is presented for the different design levels, with special emphasis placed on environmental protection issues to clarify the interfaces with the other design components and legal requirements.

In conclusion, it can be noted that the environmental compatibility examination presents an integrated procedure in road planning and design. With increasing planning accuracy, increasing verification of environmental issues takes place. The following environment-specific investigations must be performed in detail at the individual design levels according to Figure 1 (*8*).

Basic planning	Ecological risk evaluation
Alinement design	Environmental compatibility study
Detailed design elaboration	Attendant plan for landscape cultivation

Because of the close involvement between the design flow and the ECS in an integrated road design, it will become necessary to regard environmental protection issues to a greater extent in the future.

ENVIRONMENTAL COMPATIBILITY EXAMINATION IN WESTERN EUROPE: EXAMPLE GERMANY

As discussed previously, the governments of most western European countries have enacted ECE laws (*5*) and also require engineers to perform ECSs (*6*) when planning highways, railroads, waterways, and airports. These studies must always be done before starting the design phases of a project.

Thus, for major new construction, reconstruction, rehabilitation, and restoration projects, an environmentally justified compatibility study must be developed for any future highway route or location in western Europe, which will be addressed in the following discussion. The studies must include all existing natural, ecological, and cultural resources that are important to the integrity of an observed region. [These statements agree fully with the U.S. point of view (*9*)].

The space-related sensitivity investigation of the ECS is based on a presentation that covers the entire area for all relevant functions of the environment, with special regard to the following:

- Protected and protection-worthy settled and unsettled areas and resources,
- Areas with special environmental sensitivities or with specific significance for the environment, and
- Existing and planned land use.

In addition to the space-related sensitivity investigation, the highway engineer must identify alternative alinements that should be evaluated in multidisciplinary cooperation with regard to relatively low conflict corridors. By comparing the alternatives, the results must be presented on the basis of numerous elaboration steps (*5*); the most important ones are as follows:

- Evaluating the advantages and disadvantages of the alternatives,

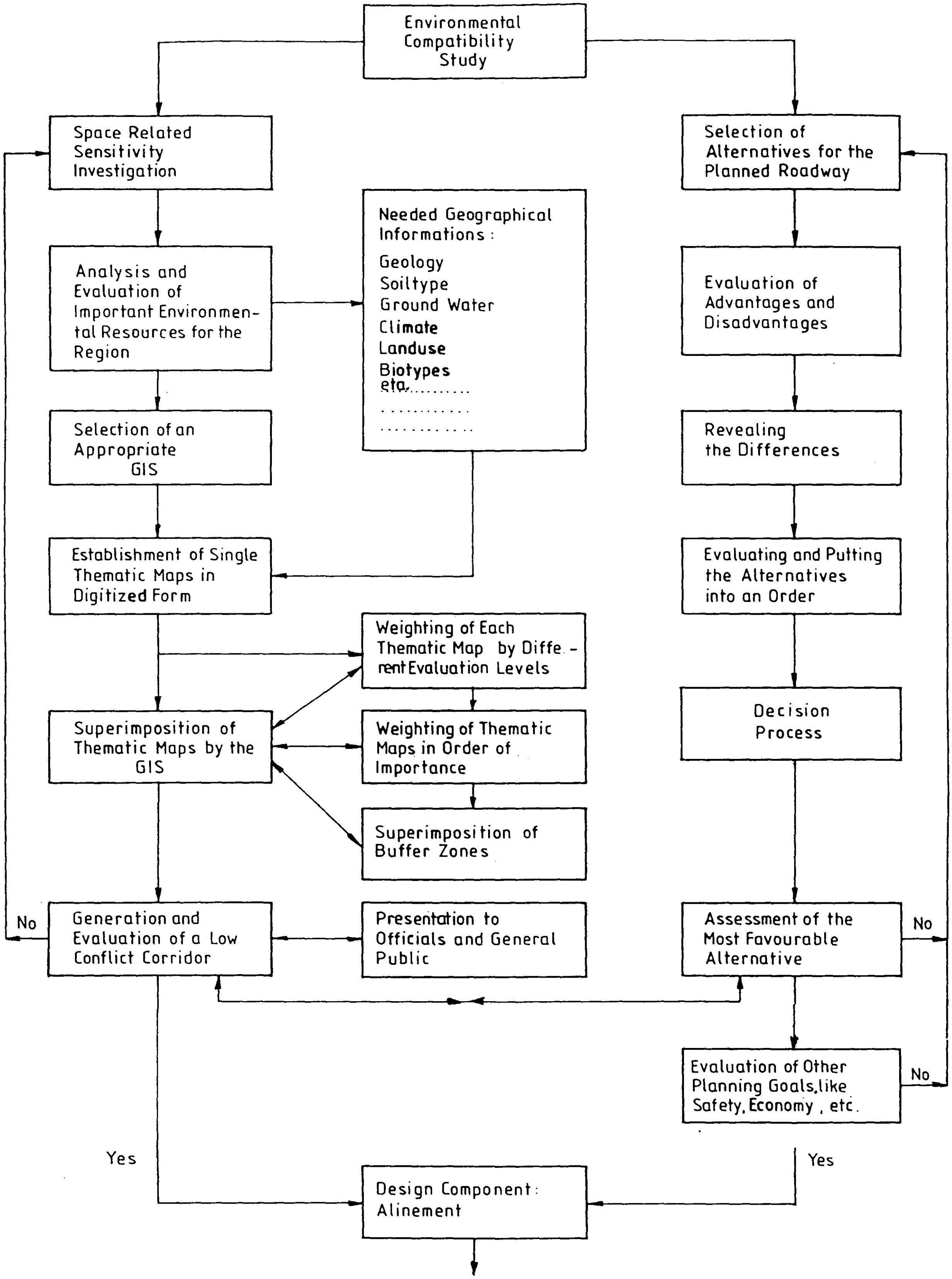

FIGURE 2 **Flowchart for the design component: environmental compatibility study.**

- Revealing the differences, and
- Evaluating the alternatives and putting them into order from the position of environmental compatibility to reach a sensible decision.

For a better understanding of the individual steps that are involved in an ECS, the flow chart in Figure 2 was developed. On the left side, the impact of the space-related sensitivity investigation is presented, whereas the right side reveals the selection of alternatives for the planned roadway. As can be seen, other important planning goals, such as safety, aesthetics, and costs, are also evaluated in connection with the assessment of the most favorable alternative.

Geographic information systems (GISs) are recommended in the *Instructural Guide for the Environmental Compatibility Study in Highway Planning* (ECS) (*6*) for conducting the sensitivity investigation. The following simplified case study reveals the application of a GIS for generating a low-conflict corridor for one alternative alinement.

CASE STUDY FOR THE DEVELOPMENT OF A LOW-CONFLICT CORRIDOR

In the varied and intensively used cultural landscape of western Europe, the sensitivity investigation will identify few areas free of conflicts that would allow for the design of new highways. Rather, a pattern will emerge in which conflicts from competing important environmental goals vary from area to area, with some areas having more impacts (conflicts) than others.

To develop a low-conflict corridor, it is necessary to search one's way through all of the conflicting environmental concerns to find those areas where environmental concerns are limited. A low-conflict corridor is an area with a relatively low level of need for environmental protection, minor environmental meaning, and a low level of environmental sensitivity where a highway between goals A and B can be designed (*5,6*); that is, it is the corridor in which a highway would have the least environmental impact.

In the center of the case study area stands the town of Staufen im Breisgau, which is in southwestern Germany, near the French and Swiss borders. The town is in a scenic and commercially rich region. The historical town center, with its medieval half-timbered houses and attractive old vineyards in the surrounding area, attracts thousands of tourists each year. The Black Forest, with its rich biotypes, is in the vicinity.

Increasing numbers of tourists and the growing economy during the past two decades have caused an enormous increase in through traffic in Staufen, with an increase in both accident frequency and severity. An important state route (SR 123) leads directly through the residential, shopping, and commercial areas of the town. Therefore, the federal, state, and municipal agencies have decided to provide a bypass around Staufen to alleviate the critical traffic conditions and safety problems.

In-depth environmental studies are required for attractive and environmentally sensitive regions like the Staufen area. To understand the important environmental impacts of the planned bypass, data bases in the form of thematic maps were developed by experts. In this connection it was found that the protection of the following environmental elements is of specific importance for the Staufen region and would need to be examined by an ECS before the acutal design and construction phases of the bypass could begin. The environmental elements to be investigated in the space-related sensitivity investigation of the ECS for the Staufen region are

1. Groundwater potential,
2. Climate,
3. Biotype distribution,
4. Land use, and
5. Recreational value.

The following environmental issues were not regarded in detail in the case study:

1. Landscape (politicians decided to avoid negative impacts on the Black Forest region east of Staufen because of its areas of natural beauty, wildlife, and recreational significance that had to be conserved; therefore, the plain adjacent to the Rhine River was the sole planning alternative);
2. Geology and soil type (in the planning area, similar strata and soil types are present; thus, the impacts of geology and soil type could be excluded from further analysis);
3. Noise (from the beginning it was decided to provide as much distance as possible between the planned bypass and residential and recreational areas and to protect the areas that would be affected by using noise protection barriers; therefore, the impact of noise was not shown on a thematic map); and
4. Others (no more relevant issues were found by the multidisciplinary team of experts studying the natural resources of the Staufen region).

To protect the first five environmental resources discussed above, thematic maps were used to decide whether specific areas are suitable for the bypass (see Figures 3 to 7). The thematic maps were developed by AKG Software Consulting, Ballrechten-Dottingen, Germany.

Regarding the time and costs required to establish the thematic maps, it should be mentioned that in Germany the time period between the stages of basic planning and approval of the plan assessment (Figure 1) is usually at least 5 years because of the complex and complicated legal procedures. In the present case study, about 1 year was needed for the basic elaboration of the thematic maps without considering the additional time that was lost for detailed analyses during the planning processes; these detailed analyses were necessary because of public and private protests. The costs up to this point can be estimated to be about 50 percent of the planning costs to this point. No indications of the portion of the overall project costs can be given, because the project has not been completed.

The data bases represented by the thematic maps in Figures 3 to 7 contain current conditions as well as future conditions concerning new residential, commercial, and recreational developments, as far as they were known during the elaboration phases.

Because geographical information based on thematic maps was available for the present study, a GIS (*10–13*) appeared to be the most suitable procedure for analyzing the complex relationships. A Canadian program known as SPANS (*14–16*) was used in the study to search for a low-conflict corridor on the basis of the five thematic maps shown in Figures 3 to 7. The benefit of SPANS is that data of different formats and from different origins can be read, analyzed, and displayed together. Normally, the thematic maps originated by a GIS are differentiated by discriminating colors. This was not possible here, so discriminating hatchings were used to present the results; these may sometimes be of lesser quality than color graphical presentations.

Thematic Maps: Groundwater Potential

The thematic map of the groundwater potential is shown in Figure 3. Discriminating hatchings are used to differentiate three main groundwater levels: low, medium, and high. If the level is low, the risk of fast and uncontrolled flow of pollutants is low. On the contrary, if expressways with high traffic loads (like the bypass in the Staufen region) are planned in areas with high groundwater levels or permeable soils, they might pose a great source of danger for the environment. Traffic accidents involving tanker trucks or other high-risk transports can happen any time and could threaten the groundwater resources because of pollution.

Therefore, the groundwater level must be regarded as an important issue for the sensitivity investigation. Areas with high groundwater levels and short distances to local drainage systems should be avoided, if possible, when planning a new highway (in this case, a four-lane divided bypass).

In connection with groundwater levels, the presence of permeable or impermeable soils is also of great importance. For example, in the case of an accident, impermeable polluted soils can be dug out to prevent groundwater pollution. Because in the Staufen region the soils are comparable and mainly impermeable, it was not necessary to include in this study the thematic map for soils.

In conclusion, high-volume roads should not be planned in areas with high groundwater levels or permeable soils (Figure 3).

Thematic Map: Climate

Thematic maps of climate may include temperature, weather, and humidity conditions as well as wind speeds and directions. Because the region to be investigated is relatively limited, significant changes related to the first three issues are not to be expected. However because of the climatic condition caused by the nearby Black Forest Mountains going over the Staufen area into the River Rhine Valley, wind speed and direction play important roles. Therefore, the thematic map of wind conditions was established and is presented in Figure 4.

For example, alterations in the morphological contours of the landscape created by building highways and railroads—elevated or depressed, including the corresponding structures—can affect local wind systems significantly. In addition, cutting down parts of forests for traffic routes could lead to wind speeds up to four times greater than those before the forests were cut. Local experts have shown that the newly planned bypass will not cause these types of major impacts in the Staufen region. Wind speed and direction, however, are always important issues in connection with every major road being planned.

The local wind speeds are arranged on the thematic map of Figure 4 and are again differentiated by three levels (high, medium, low). The main wind direction is shown by arrows. In many cases the emission concentration depends strongly on weather conditions. In particular unfavorable circumstances exist for wind calm and inversion situations. Therefore, for the same amount of emission there can exist emission concentrations that differ by a factor of 5. (The Greek capital Athens is a typical example of a location that often has extreme emission loads.)

Considering the planned bypass project, it is important that the wind force is sufficient to blow away and disperse the exhaust emissions, which mainly consist of health-damaging substances (carbon monoxide, hydrocarbons, and nitrogen monoxide from gasoline engines as well as soot particles from diesel engines). Alternatively, the main wind direction should not flow from the highway corridor to residential or recreational areas.

In conclusion, in planning the locations of high-volume roads, sufficient wind speed and the main wind direction are of great importance (Figure 4).

Thematic Map: Biotype Distribution

Biotypes are the natural areas where protected species live. These areas are very important for the ecological balance of a region.

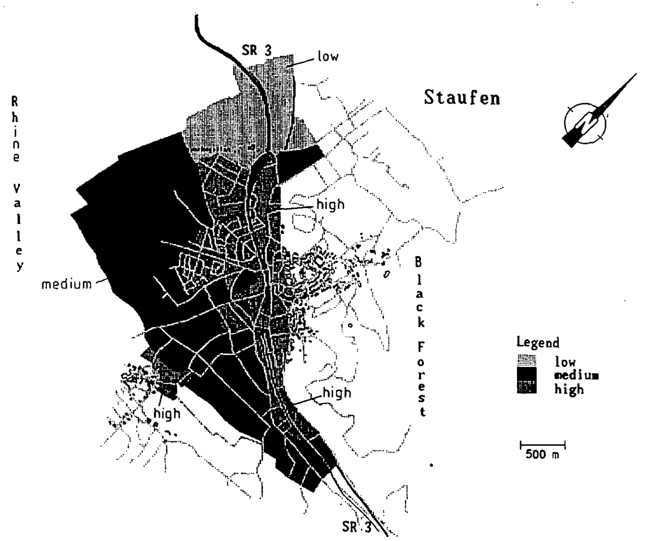

FIGURE 3 Groundwater potential.

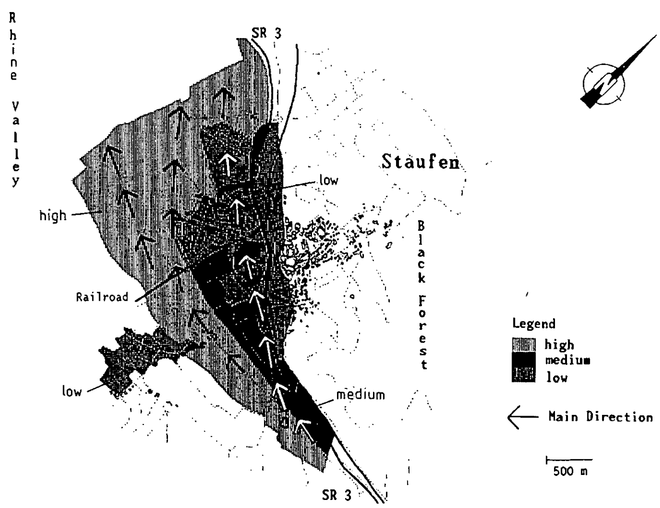

FIGURE 4 Climate (wind speed).

Biotypes include the fauna (biotypes of animals) and flora (biotypes of plants) of the observed area.

The thematic map of the biotype distribution is shown in Figure 5. The necessary data were developed and provided by a local biological expert. Again, the map is differentiated into three levels by using discriminating hatchings. *High* means conservation areas with natural beauty and wildlife: in the present case, mostly river, brook, and pond areas, parks and gardens, as well as the Black Forest region at the northeast side of Staufen. For this case study *low* means relatively invaluable agricultural and commercial land.

In conclusion, the distances between the planned bypass and important biotype areas should be as great as possible (Figure 5).

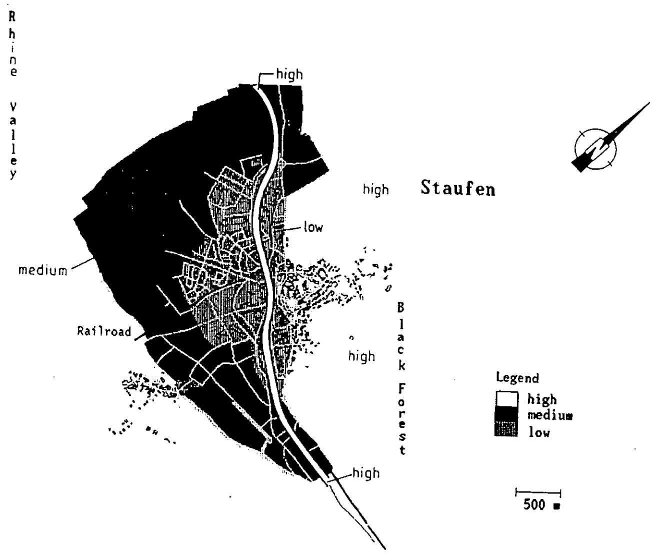

FIGURE 5 Biotype distribution.

Thematic Map: Land Use

The land use of the Staufen region is shown in the thematic map in Figure 6. As can be seen, the land use is subdivided into residential and shopping areas, commercial, and agricultural areas and green land. Politicians and local groups requested that residential and shopping areas be protected by all means.

In conclusion, for the planned bypass, agricultural and green lands as well as commercial lands can be used.

Thematic Map: Recreational Value

According to the thematic map in Figure 7, the recreational value of the area concerned can be evaluated. The map in Figure 7 was based on a poll conducted in the Staufen region in 1990. Thus, the results presented in the thematic map in Figure 7 express the opinions of Staufen citizens, visitors, and tourists.

In conclusion, only areas with mainly low levels of recreational value should be used for the planned bypass project.

Other Thematic Maps

For other planning projects, additional thematic maps are certainly of importance (*5,6*). The environmental impacts are so complex that every project usually requires a modified selection of thematic maps for conducting environmental compatibility studies in highway planning. This, however, does not change the fundamental procedure of the ECS in searching for a relatively low conflict corridor by using GISs for superimposing project-specific thematic maps, as discussed below.

Superimposition of Thematic Maps and Buffer Zones

On the basis of the five thematic maps that were developed, the space-related sensitivity investigation of the ECS will be per-

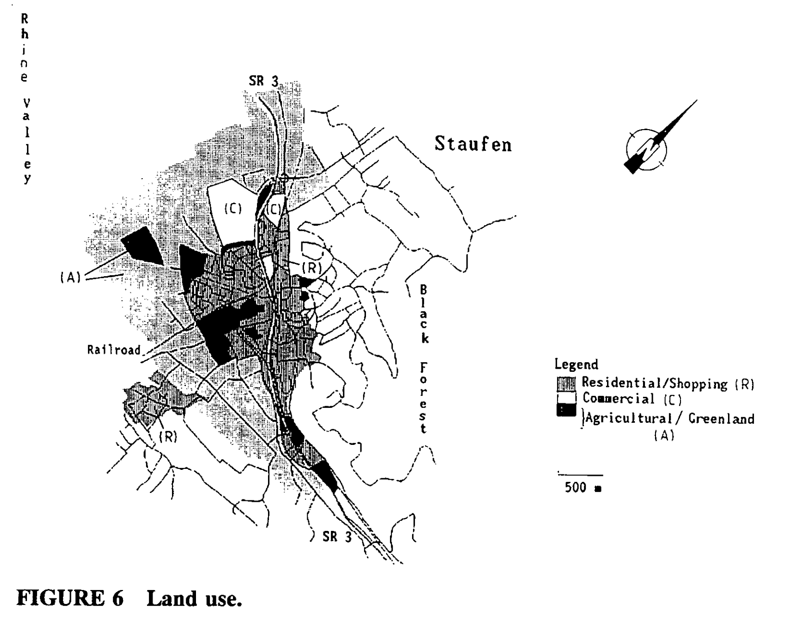

FIGURE 6 Land use.

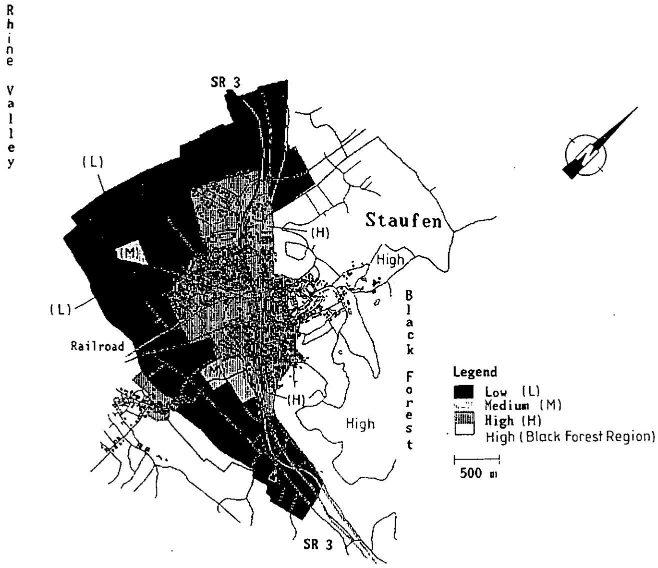

FIGURE 7 Recreational value.

formed for the planned bypass of Staufen. The analysis is done by the superimposition of the thematic maps (Figures 3 to 7) by using a computer-supported GIS examination procedure. The Canadian program SPANS (*14–16*) was used in the study. The thematic maps are available in the computer in digitized form and are therefore usable directly for the calculation processes provided in SPANS. The different layers (thematic maps) of such a map superimposition should contain all important data of the observed region to differentiate protection-worthy and less valuable areas (normally distinguished by the levels low, medium, and high). These evaluation levels, based on digitized map information, can be described on the personal computer screen or a printout by using discriminating colors or hatchings (Figures 3 to 7).

One great advantage of this method of analysis is that computer-supported GISs have the ability of calculating a weighted examination of the different superimposed digitized thematic maps. *Weighted examination* means calculation of the data from the thematic maps to create a new map that includes all relevant results of the superimposition process.

In actual practice, the weighting process is a two-step process, according to Figure 2. First, each thematic map is analyzed by the study team and subdivided into areas of similar environmental impacts. An evaluation level (for example, low, medium, or high) is then assigned to each area. The GIS allows the user to assign a weight to each evaluation level of a thematic map. In the present case study, the first step of Figure 2 ''weighting of each thematic map'' was performed by weighting the advantageous evaluation level by the factor 3, the medium level by the factor 2, and the disadvantageous evaluation level by the factor 1.

This information is entered into the GIS. The study team could also have selected numerous other factors by this procedure.

Next, the GIS can be applied to superimpose the thematic maps on each other in order of importance. This order is decided by the study team and reflects the importance of local conditions. SPANS allows this to be done sequentially. For this case study, the second step of Figure 2, weighting of thematic maps in order of importance, was performed in the following way. The most important environmentally justified issue was the groundwater potential; this was followed by the thematic maps for wind speed and direction, biotype distribution, land use, and recreational value.

According to two decision criteria, order of evaluation levels within each single thematic map and sequence of the thematic maps between each other, the computer-supported calculation procedure of SPANS generates an overall map of the results. On the basis of the results shown in the composite map, areas of different levels of environmental impact can be identified (Figure 8). By identifying the areas with environmental sensitivity, the suitability for a low-conflict corridor or corridors can be established for one or more alternatives for the planned bypass. (That means, according to Figure 8, a relatively high level of suitability should be present in the corridor.)

In addition, distance buffers around the corresponding alternatives can be superimposed on the new area-specific map. The distance buffers are provided in steps of 50 m. Thus, distance requirements can be examined by assessments such as the following:

- The new highway should pass no closer than 250 m to residential or recreational areas, or
- The distance between the bypass and a protection-worthy biotype should be at least 100 m.

Of course, the input of any other local distance requirement is possible. Those distance arrangements can be solved clearly and quickly by a GIS.

By assigning weights to the different evaluation levels of thematic maps in a reasonable order, by superimposing the thematic maps in a desired sequence, and by introducing distance buffers, a relatively low conflict corridor with lateral buffer zones could be established for the planned bypass in the Staufen region, according to Figure 8.

The overall width of the buffer zones for the established corridor covers 500 m, 250 m at each side of the axis of the planned bypass. By comparing the results of Figure 8 with the evaluation levels in Figures 3 to 7, it can be concluded that the corridor, developed by the computer-supported GIS examination procedure, excludes at least high-conflict areas.

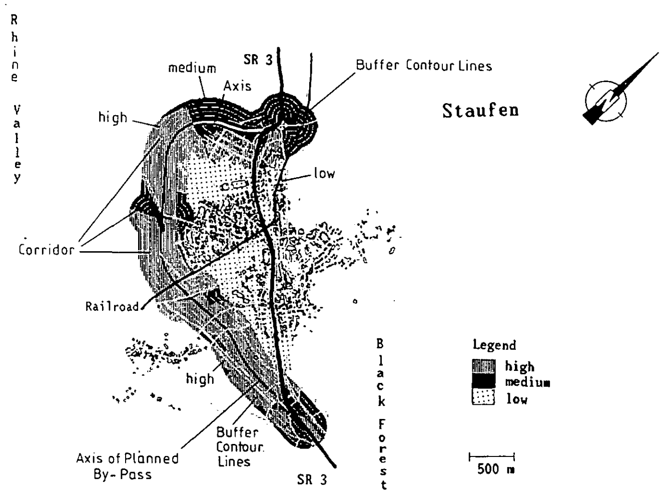

FIGURE 8 Suitability for the low-conflict corridor.

Results like those in Figure 8 will help federal, state, and municipal officials make unbiased and factually correct decisions regarding the protection worthiness of the environmental issues that are being discussed.

In addition, according to Figure 2 (right side), the ECS requires a comparison of alternatives. For the bypass around Staufen, three alternatives were investigated. The relatively low-conflict corridor shown in Figure 8 proved to be the most favorable one for the planned four-lane divided bypass according to the three sensitivity investigations conducted for the different alternatives. The evaluation must also include other important goals, aesthetics, economy, function, safety, and traffic quality. These important goals must always be kept in mind in parallel with the development of the new design component ECS (compare Figures 1 and 2).

Because the corridor lies in the plain of the Rhine River Valley, longitudinal slopes are low and considerations of vertical alinement are of minor importance. Thus, the low-conflict corridor first identified serves the placement of the horizontal alinement.

Finally, two questions should be answered. These questions mostly appear when discussing or presenting the results of an ECS:

1. Do there exist cost and time comparisons between existing methods of developing an ECS and those using a GIS? The only answer can be as follows: if an appropriate GIS is available, the cost and time factors will be reduced considerably because of the automatic weighting and evaluation processes performed on the basis of quantitative criteria (expressed, for example, by thematic maps). Only a few aspects of an ECS could be dealt with manually, if at all, and only qualitative results could be expected. A cost and time estimation would lead to speculation because of the numerous different relationships and interrelationships that would influence a specific project.
2. Would the GIS data base approach change the location of a planned highway over the location that might have been selected by engineers who may have been provided similar information but without the graphical analysis tools? The only answer can be probably not. However a question remains: How would the engineer be provided those complex pieces of information without the graphical analysis tools of a GIS?

CONCLUSION AND OUTLOOK

The design flow in modern highway geometric design guidelines with special emphasis on environmental protection issues was discussed in relation to the corresponding interfaces to other design components and legal demands.

Because most western European governments have enacted ECE laws, a new design component called ECS must be regarded in future highway geometric design guidelines.

ECS consists of two parts: (a) space-related sensitivity investigation and (b) comparison of alternatives.

ECE and the ECS already present an integrated procedure in road planning and design. With increasing planning accuracy, an increasing level of verification of environmental issues takes place.

An essential part of the ECS, the sensitivity investigation, must include all relevant functions of the environment with regard to areas worthy of protection, areas with specific sensitivities, and existing and planned land uses. GISs are recommended for use in conducting those studies. The goal of every ECS is to identify the highway corridor and the highway alternative that together have the least environmental impact on the region. A second part is concerned with the selection of alternatives for the planned roadway.

A case study involving a sensitivity investigation with thematic maps based on digitized geographical information was presented. The thematic maps were developed to portray important environmental resources, such as groundwater potential, climate, biotype distribution, and land use. By using a computer-supported GIS examination procedure, the thematic maps were superimposed, weighted, and evaluated to develop a new map. The new map, which now represents those environmental resources on a single map, is used to identify the low-conflict corridor (the corridor that minimizes environmental concerns) in the investigated region.

One important consequence of the present study is that in highway geometric design an ECS must be conducted before starting any predesign, design, or even construction phase of a project. Examination of the other important planning goals takes place after the assessment of the most favorable alternative by the ECS. The case study revealed in this way how protection-worthy areas can be left intact, and future highways can be placed in those areas for which it is less important to maintain the integrity of the region being examined. Finally, such a procedure saves money and time in the complex highway geometric design process, because only those alternatives for which mainly low-conflict areas are available and their corresponding alinements must be considered. Normally, there are only a few such areas. Thus, the optimum solution is a product of the ECS, which is conducted at the beginning of the planning process.

Incorporation of the previous concepts about the new ECS design component into a computerized GIS format is a significant step forward in accomplishing such work and in helping to present it to the public for clearer understanding, and it will probably contribute toward general public acceptance and approval.

It is recommended that future editions of the Green Book include material on the new ECS design component.

ACKNOWLEDGMENT

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Procedure for Detecting Errors in Alinement Design and Consequences for Safer Redesign

RUEDIGER LAMM, HEIKO STEFFEN, AND A. K. GUENTHER

A procedure for evaluating the horizontal alinement of two-lane rural roads on the basis of three individual safety criteria is introduced. On the basis of these criteria, design practices are classified into three groups: good, fair, and poor. The procedure can be used to identify potential safety errors in new designs already in the planning stages as well as to detect safety deficiencies in existing roadways. To be effective, the safety evaluation process must be integrated into the modern highway design tools available to highway design engineers of today. These tools consist of computer-automated design (CAD) systems for highway geometric design and normally contain a component for the design of horizontal alinement. To incorporate the safety evaluation process into the horizontal alinement component of a commonly used CAD system, a subprogram for safety computations was developed on the basis of the three individual criteria. The safety evaluation process provides a future assessment of horizontal alinement on the basis of quantitative criteria. Consequently, safety impacts can be included along with the normally considered local, environmental, esthetic, and economic aspects in making decisions on a project. A case study of an existing two-lane rural roadway in southwestern Germany is included. On the basis of safety criteria, sections of the road have poor design. The safety evaluation procedure is applied to the identification of safer redesigns. In a first step for an economical redesign, still sections with fair design practices are included. In a second step for a redesign of an overall sound curvilinear alinement, only good design practices exist.

A safety evaluation process that allows highway engineers to evaluate horizontal alinements is presented in this paper. The safety criteria are based on evaluations of complex data systems developed by the authors in cooperation with Elias M. Choueiri of the State University of New York for the United States (*1,2*) and Germany (*3,4*). Because of the available data bases, the system is applicable only for two-lane rural roads with longitudinal grades of up to 6 percent and annual average daily traffic (AADT) values of 10,000 vehicles per day.

The study is a continuation of the paper of Lamm and Smith in this Record. The model consists of three safety criteria. Safety Criterion I (achieving consistency in horizontal alinement) and Safety Criterion II (harmonizing design speed and operating speed) were already discussed thoroughly in the paper mentioned above. Safety Criteria I and II are based on operating or design speed changes between successive design elements and for single design elements to achieve good designs (for example, by sound curvilinear alinement), to classify fair designs, and to detect poor designs. The quantitative ranges for the safety evaluation process are given in Figure 1.

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To avoid repetitions, it is recommended that the reader who is interested in more detailed information about the mathematical background, analysis, development, and assessment of the ranges for Safety Criteria I and II consult the paper by Lamm and Smith in this Record and the corresponding references.

A third safety criterion regarding relevant driving dynamic aspects was basically developed previously (*5*). In this connection it was shown that the side friction factors for curve design assumed in the geometric design guidelines of AASHTO (*16*) and the German Road and Transportation Research Association (*7*) for different design speeds are often exceeded by those demanded by the 85th percentile speeds under realworld conditions. These situations begin with degrees of curve of more than 5 to 6 degrees and correspond to radii of curve of less than 350 to 290 m (1,150 to 950 ft). Furthermore, it can be proved that, in the case of good design practices, the assumed side friction exceeds the demanded side friction. In the case of poor design practices, the demanded side friction exceeds the assumed side friction.

How the geometrically assumed side friction and the demanded side friction are derived for Safety Criterion III (providing adequate dynamic safety of driving) was discussed previously (*5*) with regard to degree of curve, operating speed, and accident rate.

A first synopsis incorporating all three safety criteria into an overall safety module for evaluating road networks was presented previously (*8*). For the application of the safety module, the ranges of the driving dynamic Safety Criterion III were finally established and are shown with insignificant modifications in Figure 1. By using a geographical information system (GIS) in connection with the developed safety module, the designer can immediately recognize different design safety levels (good, fair, poor) by discriminating colors or symbols at the PC screen or on printouts (*8*).

Such a procedure is ideal for obtaining a fast overview of the safety situation of whole or partial road networks, including the combined results of all three so far equally weighted safety criteria. Although the overview is useful, corrective action by the highway engineer requires knowledge of the specific deficiencies for each highway section. Therefore, in cases of new designs, redesigns, and resurfacing, restoration, or rehabilitation or (RRR) projects of specific roadway sections, all three safety criteria must be analyzed individually.

On the basis of the different safety aspects, the results of the three safety criteria in Figure 1 do not always agree. For example:

- A curved section may be classified by Safety Criteria I and II as "good." That would mean the absolute differences between

CRITERION	GOOD	FAIR	POOR
	DESIGN PRACTICES		
I	$\lvert V85_i - V85_{i+1} \rvert \leq 10$ km/h	10 km/h $< \lvert V85_i - V85_{i+1} \rvert \leq 20$ km/h	20 km/h $< \lvert V85_i - V85_{i+1} \rvert$
II	$\lvert V85 - V_d \rvert \leq 10$ km/h	10 km/h $< \lvert V85 - V_d \rvert \leq 20$ km/h	20 km/h $< \lvert V85 - V_d \rvert$
III	$0 \leq f_{RA} - f_{RD}$	$-0{,}02 \leq f_{RA} - f_{RD} < 0$	$f_{RA} - f_{RD} < -0{,}02$

V85 = 85th Percentile Speed ; V_d = Design Speed
f_{RA} = Side Friction "Assumed" ; f_{RD} = Side Friction "Demand"

FIGURE 1 Ranges of safety criteria for good, fair, and poor design practices.

the 85th percentile speeds of preceding and succeeding design elements (Safety Criterion I) as well as the absolute difference between the 85th percentile speed and the design speed in the curve itself (Safety Criterion II) would fall into the range of ≤ 10 km/hr. Despite these results, Safety Criterion III may reveal driving dynamic deficiencies, since the superelevation rate in the observed curve is too low, for example; or

- It is possible that Safety Criterion I represents a good safety level for a longer roadway section, whereas Safety Criterion II reveals that fair or even poor design practices exist because of differences between expected 85th percentile speeds and the selected design speed that are too large (Figure 1).

Because of these discrepancies, an individual examination of specific roadway sections on the basis of the three safety criteria makes more sense, contrary to the evaluation of whole road networks by a combined safety module (*8*). This is especially true when the highway engineer has information about the planned or the existing highway, the safety quality (good or fair) to be strived for, and local conditions and available funds. For example, the designer may be able to improve the alinement, in the case of a failure of only one safety criterion, in such a way that the safety deficiency can be eliminated without affecting the other criteria and their impacts on the design.

Note that besides the normal case of a speed that is too low, it is quite possible to select a design speed that is too high. In such a case, the superior goal of safety is of minor importance. However the function of the highway in the road network or the desired traffic quality may cause the design to be uneconomical (see the papers by Lamm and Smith and Lamm et al., this Record).

To recognize safety errors in new designs, redesigns in the planning stages, or designs for necessary safety improvements for RRR projects before implementation, modern planning tools must be made available to the highway engineer. Complex data processing systems must be part of today's planning tools. They are able to support the design and construction of roads beginning with environmental compatibility studies (see paper by Lamm et al., this Record); this is followed by the design processes and continues through the construction phases. Therefore, it would be of great advantage to incorporate an additional subprogram into such a planning system based on the safety evaluation processes of the three individual safety criteria discussed previously. This would allow safety errors in the alinements of new designs and deficiencies in the alinements of existing roadways to be detected and eliminated concurrently with the design or redesign processes.

According to the call for papers for the conference session on which this Record is based, the most recent AASHTO geometric design policy should be addressed. New developments not cited in the Green Book (*6*) are not included. It is extremely difficult, especially for foreign authors, to stay informed about new developments until they are included in the national standards.

FUNDAMENTALS FOR COMPUTER-AIDED HIGHWAY DESIGN

Modern data processing systems for traffic routes should consist of at least the following components:

- Environmental compatibility study,
- Geometric surveying,
- Horizontal alinement,
- Vertical alinement,
- Cross section,
- Graphical layouts (as direct derivations of the computations),
- Three-dimensional evaluation (perspective view), and
- Different construction components.

For the present study the horizontal alinement component is of special interest. Programs for the numerical computation of road axes for horizontal alinements have existed since the 1960s and were first developed by IBM (*9,10*). These programs were related to mainframe computer applications that computed whole systems of roads and interchanges on the basis of descriptive data by using explicitly provided input data. The input data, coordinates of certain fixed points, and basic information about circular and transition curves according to a predesign of the horizontal alinement are provided by the highway engineer and are based on preliminary work on location. The computer then prints out all necessary numerical design data for establishing the future road axes.

However, numerical printouts are difficult to work with, and examination of the results is nearly impossible. Therefore, modern data processing systems have the capability of providing information at both the numerical level and the graphical level, which allows for the immediate change of computational results into graphical layouts or vice versa at the PC screen or on printouts, allowing exact computations and information-control graphics to stand side by side (*11*).

The horizontal alinement component of the commonly used German computer-aided design (CAD) system was selected (*12*) for the possible integration of the new subprogram for evaluating horizontal alinement with the three individual safety criteria. With this component the axes of horizontal alinement can be computed and displayed on a PC screen for various alternatives. This is important not only for studying topographical and local conditions (see, for example, the development of a low-conflict corridor in the paper by Lamm et al. in this Record) but also for making the necessary alinement changes required by the safety evaluation process. Furthermore, all necessary design data for the axis are available in a computer-justified (digital) mode for future processing steps.

It makes sense, therefore, to develop an additional subprogram for the new safety evaluation process on the basis of the three individual criteria and to integrate this into the horizontal alinement component of an overall CAD system. Figure 2 shows the iterative flow of information. In this way the future axes of a specific roadway section could be evaluated automatically. For such a system it is not relevant whether the descriptive input data result from a new design or are related to an existing roadway.

DEVELOPMENT OF A SUBPROGRAM FOR SAFETY CALCULATIONS

Because the subprogram for safety calculations needs only the information about the geometry of the road in a computer-justified (digital) mode, this subprogram can be integrated into any CAD system for highway geometric design. The only assumption is that the system provides a clear data interface for the output of the horizontal design data in digital mode. The flow chart in Figure 2 shows that the input of the descriptive design data is possible for planned or existing roadways. As input data, the safety computation subprogram needs the geometric output data and the elements of the horizontal alinement component, which are as follows:

- Kind of design elements (curve, clothoid, tangent),
- Length of elements,
- Parameters of design elements (radius of curve, parameter of clothoid), and
- Stations.

For the safety computations, the following input data are required:

- Design speed (V_d),
- Pavement (lane) width (LW),
- Superelevation rates (e), and
- Length of independent tangents (TL) (*13*).

Tangents must be defined as *independent* and *nonindependent*. Independent tangents may cause critical changes in the operating speed profile [85th percentile speed (V_{85})] and must be regarded in the design process, whereas nonindependent tangents do not need to be regarded. In this connection the consideration of tangents as dynamic (speed-dependent) elements similar to curves is very important for the evaluation of (speed) transitions between successive design elements [for example, curve to tangent or curve to curve (*13*)].

On the basis of these input data, the relevant design parameters, degree of curve (DC) for the United States and curvature change rate (CCR) for Germany, can be determined (see paper by Lamm and Smith, in this Record). These design parameters are important for estimating the expected V_{85} and the values for side friction assumed (f_{RA}) and side friction demand (f_{RD}) needed for the safety evaluation process shown in Figure 1. The mathematical equations for the relationships between these variables were previously developed by the authors for the United States and Germany. [Readers who are interested in a detailed discussion of the derivations of those equations and the assessments for the design ranges of the safety criteria should consult previous reports for the United States (*1,2,5,8,13–15*; see also the paper by Lamm and Smith, in this Record) and Germany (*3,4,16,17*)].

All pertinent equations, as well as the ranges for the three safety criteria in Figure 1, for evaluating good, fair, and poor design practices are contained in the subprogram safety computations (Figure 2). The process described in Figure 2 is an iterative one. Therefore, an automatic safety evaluation process with regard to the input data listed above is possible for planned or existing road axes.

If this evaluation process does not reveal errors or deficiencies, the following highway geometric design procedure can be pursued. If one or more of the three individual criteria are not fulfilled, however, various design alternatives are evaluated until a satisfactory road axis is established. The procedure will be used in a case study in Germany in the following section and is based on the German assumptions for the relationships discussed earlier (*4,16,17*; see also Lamm and Smith, this Record). Therefore, Figures 3 and 4 in the paper by Lamm and Smith, this Record, are relevant to this case study.

SAFETY EVALUATION FOR THE CASE STUDY

The existing horizontal alinement in Figure 3(*a*) shows a two-lane rural state route in southwestern Germany in the plain of the Rhine River. Accident analysis indicates a high accident frequency and severity at Element 2. The longitudinal grades are less than 2

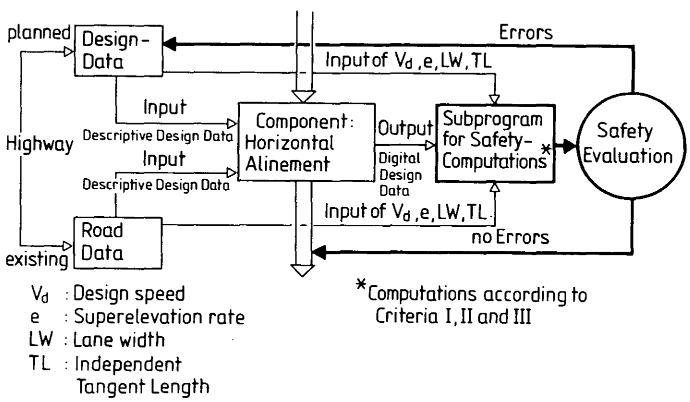

FIGURE 2 Flowchart for highway geometric design with special regard to a safety evaluation process.

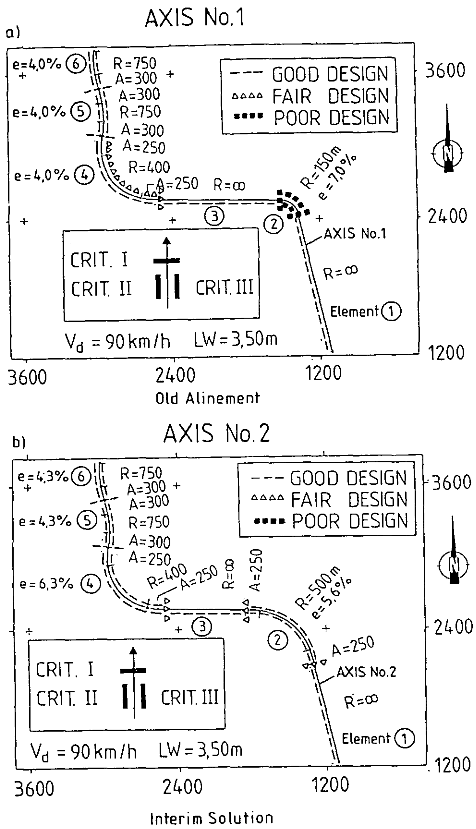

FIGURE 3 Graphical presentation of the safety evaluation process.

percent and the AADT values corresponded to 7,200 vehicles per day in 1991. The old alinement should be improved, and the new alinement should represent the level of good design practice for all three individual safety criteria. Between the stages old and new an interim solution should also be planned in the event that federal funds cannot be provided in full.

Old Alinement

Figure 3(*a*) shows the existing old alinement (Axis No. 1), which was designed in the 1930s. The lane width is 3.50 m, and the original design speed is unknown. A serious accident situation exists in the curve of design element 2 (R = 150 m), which is situated between two long independent tangents (Elements 1 and 3). Sixteen run-off-the-road type accidents occurred from 1989 to 1991; these included 3 fatalities, 6 seriously injured individuals, and 13 lightly injured individuals. The main accident cause, recorded by the police, was "improper speed estimation" in the transition sections and in the curve itself. The main goal, therefore, had to be a reduction in accident severity by appropriately redesigning the old alinement. It is interesting that for the long tangent Sections 1 and 3, no relevant accidents (for example, because of passing vehicles) were recorded.

With the exception of Element 2, all the other curved roadway sections (Elements 4 to 6) corresponded at least to a design speed of 90 km/hr according to the German *Guidelines for the Design of Roads* (*7*). Consequently, it was decided to select 90 km/hr as the design speed to keep the reconstruction costs as low as possible.

The descriptive design data for the old alinement, the design speed of V_d = 90 km/hr the lane width (LW) = 3.50 m, the measured superelevation rates, and the lengths of the independent tangents (Elements 1 and 3) of the old road represented the input data. These data are used for the horizontal alinement component and for the corresponding safety computation subprogram discussed previously and presented in Figure 2.

The output data for the safety evaluation process are listed in numerical mode in Table 1. Table 1 shows the point, made earlier in this paper, that numerical data are difficult to describe and those listings—as valuable as they are for an exact evaluation overview—may be too complex for fast and easy understanding. An analysis of the critical curve (Element 2) indicates that the absolute V_{85} differences between Elements 1 and 2 as well as between Elements 2 and 3 exceed 20 km/hr and reveal poor design according to the ranges of Safety Criterion I in Figure 1. The same is true for Safety Criterion II regarding the absolute difference between V_{85} and design speed and for the driving dynamic Safety Criterion III regarding the difference between the assumed side friction (f_{RA}) and the demanded side friction (f_{RD}) for curve Element 2. (Note that the V_{85} computed automatically by the subprogram on the basis of the CCR values could have been determined from Figure 4 of the paper by Lamm and Smith, this Record, in the case of a manual safety evaluation process.)

A graphical presentation of the numerical results in Table 1 was developed and is presented in Figure 3(*a*); the results can be used at the PC screen or printed out. In this way the different design levels, based on individual Safety Criteria I to III, can be recognized visually by using discriminating colors or symbols. For a better understanding it should be mentioned that the colors or graphical symbols (as in the present case) for Safety Criterion I are arranged vertically to be the road axis, whereas the symbols for Safety Criterion II are located on the left side and those for Safety Criterion III are located on the right side, parallel to the axis.

By evaluating the graphical layout of Figure 3(*a*) it can be recognized at once that the critical curve (element 2) corresponds to poor design practices regarding all investigated safety criteria. This result supports the previous statements about the serious accident situation at this curve site.

In addition, it can be seen that the curve with the radius of 400 m (Element 4) can be evaluated only as a fair design for Safety Criterion I when considering the transition between Elements 3 and 4. Fair design practice could also be noticed for Safety Criterion III in this curve (compare Table 1). The accident situation at this site consisted of one serious, three light, and two property damage accidents during the time period investigated, which supports the findings of the safety evaluation process.

All the other road sections of the existing alinement reveal good design practices and do not need any changes in future redesigns.

Interim Solution

A fair design practice as the minimum requirement for an interim solution was requested for the present case study to keep down the reconstruction costs. Therefore, Figure 3 of the paper by Lamm and Smith, in this Record, was referred to. Related to the critical radius of the curve (Element 2), it was found that to com-

TABLE 1 Numerical Output Data for the Safety Evaluation Process (Old Alinement)

AXIS : 1

ELEM. : 1

RADIUS	STATION FROM	STATION TO	CLOTHOIDS BEFORE	CLOTHOIDS BEHIND	CCR	V85	SUPER-ELEVATION
0	0.00	1190.42	0.00	0.00	0.00	99.70	2.5

CRIT. II : $|V85_1 - V_d|$ = 9.70 => GOOD DESIGN

Transition 1-2 for Crit. I : $|V85_1 - V85_2|$ = 32.98 => POOR DESIGN

ELEM. : 2

RADIUS	STATION FROM	STATION TO	CLOTHOIDS BEFORE	CLOTHOIDS BEHIND	CCR	V85	SUPER-ELEVATION
-150	1190.42	1390.00	0.00	0.00	424.67	67.32	7.0

CRIT. II : $|V85_2 - V_d|$ = 22.68 => POOR DESIGN
CRIT. III : $f_{RA} - f_{RD}$ = -0.09 => POOR DESIGN

Transition 2-3 for Crit. I : $|V85_2 - V85_3|$ = 32.98 => POOR DESIGN

ELEM. : 3

RADIUS	STATION FROM	STATION TO	CLOTHOIDS BEFORE	CLOTHOIDS BEHIND	CCR	V85	SUPER-ELEVATION
0	1390.00	2373.79	0.00	0.00	0.00	99.70	2.5

CRIT. II : $|V85_3 - V_d|$ = 9.70 => GOOD DESIGN

Transition 3-4 for Crit. I : $|V85_3 - V85_4|$ = 15.95 => FAIR DESIGN

ELEM. : 4

RADIUS	STATION FROM	STATION TO	CLOTHOIDS BEFORE	CLOTHOIDS BEHIND	CCR	V85	SUPER-ELEVATION
400	2373.79	3195.87	250.00	-250.00	128.98	83.75	4.0

CRIT. II : $|V85_4 - V_d|$ = 6.25 => GOOD DESIGN
CRIT. III : $f_{RA} - f_{RD}$ = -0.02 => FAIR DESIGN

Transition 4-5 for Crit. I : $|V85_4 - V85_5|$ = 7.66 => GOOD DESIGN

ELEM. : 5

RADIUS	STATION FROM	STATION TO	CLOTHOIDS BEFORE	CLOTHOIDS BEHIND	CCR	V85	SUPER-ELEVATION
-750	3195.87	3586.17	300.00	-300.00	58.82	91.41	4.0

CRIT. II : $|V85_5 - V_d|$ = 1.41 => GOOD DESIGN
CRIT. III : $f_{RA} - f_{RD}$ = 0.03 => GOOD DESIGN

Transition 5-6 for Crit. I : $|V85_5 - V85_6|$ = 1.25 => GOOD DESIGN

ELEM. : 6

RADIUS	STATION FROM	STATION TO	CLOTHOIDS BEFORE	CLOTHOIDS BEHIND	CCR	V85	SUPER-ELEVATION
750	3586.17	3906.89	300.00	0.0	69.04	90.16	4.0

CRIT. II : $|V85_6 - V_d|$ = 0.16 => GOOD DESIGN
CRIT. III : $f_{RA} - f_{RD}$ = 0.03 => GOOD DESIGN

Legend: CCR = German design parameter "Curvature Change Rate", compare Figure 4 in paper by Lamm and Smith, in this Record.

TABLE 2 Numerical Output Data for the Safety Evaluation Process (Interim Solution)

AXIS : 2

ELEM. : 1

RADIUS	STATION FROM	STATION TO	CLOTHOIDS BEFORE	CLOTHOIDS BEHIND	CCR	V85	SUPER-ELEVATION
0	0.00	852.31	0.00	0.00	0.00	99.70	2.5

CRIT. II : $|V85_1 - V_d|$ = 9.70 => GOOD DESIGN

Transition 1-2 for Crit. I : $|V85_1 - V85_2|$ = 13.80 => FAIR DESIGN

ELEM. : 2

RADIUS	STATION FROM	STATION TO	CLOTHOIDS BEFORE	CLOTHOIDS BEHIND	CCR	V85	SUPER-ELEVATION
-500	852.31	1642.60	250.00	-250.00	107.25	85.90	5.6*

CRIT. II : $|V85_2 - V_d|$ = 4.10 => GOOD DESIGN
CRIT. III : $f_{RA} - f_{RD}$ = 0.02 => GOOD DESIGN

Transition 2-3 for Crit. I : $|V85_2 - V85_3|$ = 13.80 => FAIR DESIGN

ELEM. : 3

RADIUS	STATION FROM	STATION TO	CLOTHOIDS BEFORE	CLOTHOIDS BEHIND	CCR	V85	SUPER-ELEVATION
0	1642.60	2288.28	0.00	0.00	0.00	99.70	2.5

CRIT. II : $|V85_3 - V_d|$ = 9.70 => GOOD DESIGN

Transition 3-4 for Crit. I : $|V85_3 - V85_4|$ = 15.95 => FAIR DESIGN

ELEM. : 4

RADIUS	STATION FROM	STATION TO	CLOTHOIDS BEFORE	CLOTHOIDS BEHIND	CCR	V85	SUPER-ELEVATION
400	2288.28	3110.37	250.00	-250.00	128.98	83.75	6.3*

CRIT. II : $|V85_4 - V_d|$ = 6.25 => GOOD DESIGN
CRIT. III : $f_{RA} - f_{RD}$ = 0.00 => GOOD DESIGN

Transition 4-5 for Crit. I : $|V85_4 - V85_5|$ = 7.66 => GOOD DESIGN

ELEM. : 5

RADIUS	STATION FROM	STATION TO	CLOTHOIDS BEFORE	CLOTHOIDS BEHIND	CCR	V85	SUPER-ELEVATION
-750	3110.37	3500.66	300.00	-300.00	58.82	91.41	4.3*

CRIT. II : $|V85_5 - V_d|$ = 1.41 => GOOD DESIGN
CRIT. III : $f_{RA} - f_{RD}$ = 0.03 => GOOD DESIGN

Transition 5-6 for Crit. I : $|V85_5 - V85_6|$ = 1.25 => GOOD DESIGN

ELEM. : 6

RADIUS	STATION FROM	STATION TO	CLOTHOIDS BEFORE	CLOTHOIDS BEHIND	CCR	V85	SUPER-ELEVATION
750	3500.66	3821.38	300.00	0.0	69.04	90.16	4.3*

CRIT. II : $|V85_6 - V_d|$ = 0.16 => GOOD DESIGN
CRIT. III : $f_{RA} - f_{RD}$ = 0.04 => GOOD DESIGN

* Calculated value should be rounded for the construction process.

bine a tangent and a curve in the fair design range the least possible radius is R 500 m; see Axis No. 2 in Figure 3(*b*). Furthermore, the authors decided to apply the exact superelevation rates provided by the German guidelines (*7*). For the same safety evaluation procedure, as discussed before but based this time on the descriptive design data for Axis No. 2 and the other relevant input data (such as the same design speed, lane width, superelevation rates, and tangent lengths), the results are listed in Table 2 and shown graphically in Figure 3(*b*). As can be seen, the interim solution reveals fair design practices between Elements 1 to 2, 2 to 3, and 3 to 4. That means, in relation to Safety Criterion I, the absolute differences in the V_{85}s for these element sequences lie somewhere in the range of between 10 and 20 km/hr according to Figure 1. Safety Criterion II and III represent, with no exception, good design practices.

From an economical point of view the alinement in Figure 3(*b*) can be evaluated as favorable because of low construction costs (at least 50 percent less than those for the final curvilinear alinement). However, it is difficult to determine to what extent the remaining transition sections with fair designs may have an unfavorable impact on the accident situation. As a matter of fact, however, for the section with a fair design, higher accident risks can be expected than on sections with good designs (*1,2,5,15*).

Final Curvilinear Alinement

For safety reasons, good design practices should always be strived for if no other superior goals are of relevant importance. This is true for the new design of multilane as well as two-lane rural roads. Besides the individual Safety Criteria I to III discussed here, one tool for achieving good designs is introduced by the term *curvilinear alinement* or *relation design* in the paper by Lamm and Smith, this Record. This means that single design elements should no longer be put together; rather sound design element sequences should be formed. To support this idea, relationships for the tuning of sound radii of curve sequences were developed in Figure 3 for Germany and in Figure 6 for the United States in the paper by Lamm and Smith, this Record.

For the following relation design in the present case study, the German assumptions were again taken as the basis, and only radii of curves between successive design elements that fell at least into the good range of the above-mentioned diagram (Figure 3 and the paper by Lamm and Smith, this Record) were selected. The resulting curvilinear alinement is shown as Axis No. 3 in Figure 4(*a*). The results of the safety evaluation process according to Table 3 and Figure 4(*a*) show no safety errors or deficiencies on the basis of Safety Criteria I to III. All three criteria confirm good design practices for the curvilinear alinement along the whole two-lane rural roadway section. Thus, it can be expected that the final alinement, presented in Figure 4(*a*) is a sound one.

Other Aspects

It can now be observed that by eliminating the tangent sections a well-balanced curvilinear alinement would result and the risk of run-off-the-road accidents may be reduced.

However, by eliminating the tangents the risk of critical passing maneuvers may increase. Safe passing maneuvers require minimum passing sight distances (PSDs). Therefore, a PSD analysis

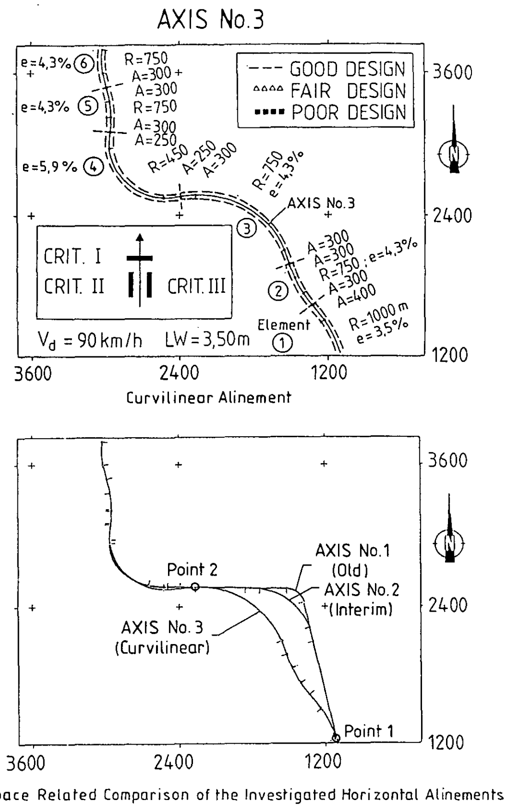

FIGURE 4 Graphical presentation of the safety evaluation process and comparison.

was conducted on the basis of a minimum PSD of 575 m, which is required for a design speed of 90 km/hr in the German design guidelines (*7*). This analysis involved the roadway from Point 1 to Point 2 in Figure 4(*b*), where the main redesign measures will take place. The rest of the alinement remains more or less unchanged. The result of the PSD analysis revealed for the observed road section that the minimum PSD always exists because of the presence of a large radius of curve between 750 and 1000 m of Axis No. 3. It could even be proved that the PSD requirements are improved decisively by the curvilinear alinement in comparison with the old alinement of Axis No. 1, in which the radius of 150 m may have had an unfavorable influence on the PSD. This is an additional positive aspect resulting from the analysis of road sections by using the three safety criteria. Therefore, negative impacts on traffic safety are not to be expected for the final curvilinear alinement resulting from PSD considerations.

The lateral displacement of Axis No. 3 in comparison with that of Axis No. 1 [see Figure 4(*b*)] is of minor importance, because an environmental compatibility study done as described in the paper by Lamm et al., this Record, revealed that all three axes are located in a low-conflict corridor. Regarding land use, sufficient agricultural land and green land are available for the new corridor (classified as being worthy of a low level of protection), whereas the topography in the plain of the Rhine River plays an inferior role.

APPLICATION FOR THE GREEN BOOK

The introduction of a safety evaluation process for differentiating different design levels (for example, good, fair, and poor) of hor-

TABLE 3 Numerical Output Data for the Safety Evaluation Process (Curvilinear Alinement)

AXIS : 3

ELEM. : 1

RADIUS	STATION FROM	STATION TO	CLOTHOIDS BEFORE	CLOTHOIDS BEHIND	CCR	V85	SUPER-ELEVATION
-1000	0.00	449.12	0.00	-400.00	52.35	92.23	3.5

CRIT. II : $|V85_1 - V_d|$ = 2.23 => GOOD DESIGN
CRIT. III : $f_{RA} - f_{RD}$ = 0.05 => GOOD DESIGN

Transition 1-2 for Crit. I : $|V85_1 - V85_2|$ = 0.78 => GOOD DESIGN

ELEM. : 2

RADIUS	STATION FROM	STATION TO	CLOTHOIDS BEFORE	CLOTHOIDS BEHIND	CCR	V85	SUPER-ELEVATION
750	449.12	834.21	300.00	-300.00	58.47	91.45	4.3*

CRIT. II : $|V85_2 - V_d|$ = 1.45 => GOOD DESIGN
CRIT. III : $f_{RA} - f_{RD}$ = 0.03 => GOOD DESIGN

Transition 2-3 for Crit. I : $|V85_2 - V85_3|$ = 2.13 => GOOD DESIGN

ELEM. : 3

RADIUS	STATION FROM	STATION TO	CLOTHOIDS BEFORE	CLOTHOIDS BEHIND	CCR	V85	SUPER-ELEVATION
-750	834.21	1981.97	300.00	-300.00	76.05	89.33	4.3*

CRIT. II : $|V85_3 - V_d|$ = 0.67 => GOOD DESIGN
CRIT. III : $f_{RA} - f_{RD}$ = 0.04 => GOOD DESIGN

Transition 3-4 for Crit. I : $|V85_3 - V85_4|$ = 4.77 => GOOD DESIGN

ELEM. : 4

RADIUS	STATION FROM	STATION TO	CLOTHOIDS BEFORE	CLOTHOIDS BEHIND	CCR	V85	SUPER-ELEVATION
450	1981.97	2923.83	250.00	-250.00	120.68	84.55	5.9*

CRIT. II : $|V85_4 - V_d|$ = 5.45 => GOOD DESIGN
CRIT. III : $f_{RA} - f_{RD}$ = 0.01 => GOOD DESIGN

Transition 4-5 for Crit. I : $|V85_4 - V85_5|$ = 6.97 => GOOD DESIGN

ELEM. : 5

RADIUS	STATION FROM	STATION TO	CLOTHOIDS BEFORE	CLOTHOIDS BEHIND	CCR	V85	SUPER-ELEVATION
-750	2923.83	3301.17	300.00	-300.00	57.92	91.52	4.3*

CRIT. II : $|V85_5 - V_d|$ = 1.52 => GOOD DESIGN
CRIT. III : $f_{RA} - f_{RD}$ = 0.03 => GOOD DESIGN

Transition 5-6 for Crit. I : $|V85_5 - V85_6|$ = 1.37 => GOOD DESIGN

ELEM. : 6

RADIUS	STATION FROM	STATION TO	CLOTHOIDS BEFORE	CLOTHOIDS BEHIND	CCR	V85	SUPER-ELEVATION
750	3301.17	3621.89	300.00	0.0	69.04	90.16	4.3*

CRIT. II : $|V85_6 - V_d|$ = 0.16 => GOOD DESIGN
CRIT. III : $f_{RA} - f_{RD}$ = 0.04 => GOOD DESIGN

* Calculated value should be rounded for the construction process.

izontal alinement on the basis of the three individual safety criteria discussed here is recommended. The procedure should first be adjusted to the new designs and redesigns of two-lane rural roads because of the serious accident situation observed on this part of the road network. The safety evaluation process should then be incorporated into the horizontal alinement component of an appropriate CAD system for highway geometric design.

In this way it is possible to evaluate safety impacts for the future assessment of horizontal alinements by the use of quantitative criteria, in addition to the normally considered local, environmental, esthetic, and economic criteria.

CONCLUSION

A procedure for enabling highway engineers to evaluate the horizontal alinements of two-lane rural roads by applying three individual safety criteria was presented in this paper.

To recognize safety errors in new designs or redesigns in the planning stages or necessary improvements in RRR projects before implementation, modern planning tools like CAD systems for highway geometric design had to be made available. In this connection for the horizontal alinement component of the overall CAD system, an additional subprogram for a new safety evaluation process was developed. The new subprogram allows for the evaluation of the horizontal alinements of planned or existing roadways on the basis of good, fair, and poor design practices.

In this way it is possible to evaluate safety impacts for the future establishment of horizontal alinement alternatives. This allows change not only from a design point of view but also from a safety point of view.

The procedure was examined by changing the alinement of an existing two-lane rural roadway, which revealed poor design practices, via a fair but economical solution into a sound curvilinear alinement representing only good design levels.

The next research step should be to examine the validity of the results of the proposed safety model with the actual accident situation, for example, to extend the model for hazard rating or estimating the numbers of accidents (classified by rate or severity) for the road segment being considered. First efforts in this direction were made previously (*8*) and revealed good agreement. A statistically sound analysis and evaluation, however, has so far not been possible because of the present insufficient accident data bases, especially regarding single roadway sections with relatively low numbers of accidents. At present corresponding research studies are in the stage of development, and reliable comparative results may be expected in 1995. It should not be forgotten, however, that the ranges of validity according to Figure 1 were established for Safety Criterion I on the basis of mean accident rates (*1,2,15*) and for Safety Criterion III on the basis of in-depth accident investigations (*5,8*).

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Coordination of Horizontal and Vertical Alinement with Regard to Highway Esthetics

BOB L. SMITH AND RUEDIGER LAMM

The vertical and horizontal designs of highways should have a pleasing appearance when combined together. They should also fit gracefully into their surroundings and become acceptable components of the landscape as viewed from outside the highway. The coordination or proper fitting together of the horizontal and the vertical alinements is an important technique for achieving an esthetically pleasing highway alinement design. Even though the safety benefits of esthetically pleasing highways have not been well documented in the past, the literature contains statements about the subtle interrelationship between highway esthetics and highway safety; that is, those things that make a highway beautiful also can make it safer for traffic. In addition, an esthetically pleasing highway also appears to be safer to the users, which is important for their enjoyment of that highway. Definitive guidelines for safe and esthetically pleasing three-dimensional alinements are presented. The recommended practices of both Germany and the United States are discussed and compared. A safety evaluation process called *curvilinear alinement* and its application to superimposed vertical and horizontal alinements are discussed. Several specific items related to highway esthetics and related safety issues are recommended for consideration for inclusion in the next AASHTO policy on geometric design of highways and streets (Green Book).

The essential form of highways expresses their function, which is to move people and goods safely and rapidly from one place to another. Highways should have a pleasing appearance. They should fit gracefully into their surroundings and become acceptable components of the landscape as viewed from outside the highway (*1*). The coordination or proper fitting together of the horizontal and vertical alinements is an important technique for achieving an esthetically pleasing highway design.

In the United States highway esthetics is generally considered a desirable goal in design because anything worth doing is worth doing well. The safety benefits of esthetically pleasing highways have not been well quantified; nonetheless, in *Practical Highway Esthetics* (*1*), it is stated that there is a subtle interrelationship between highway esthetics and highway safety. Measures, such as smooth continuous alinement, wide recovery areas, broad rounded ditches, flat slopes, and erosion control, all of which make a highway beautiful, also make it safer for traffic. Not only is the highway actually safer but it also appears to be safer to the driver and passengers, which is important to their enjoyment of the roadsides and the scenery. *Practical Highway Esthetics* (*1*) also makes the case for "safety in variety"; that is, monotony is the enemy both of good esthetics and safe operation, and it dulls the enjoyment of the visual experience and diminishes the alertness that is essential for safe driving.

The 1990 Green Book (*2*) promotes the concept of alinement coordination principally for its esthetic value. In addition, it mentions that excellence in design owing to the coordination of vertical and horizontal alinements increases usefulness and safety, encourages uniform speed, and improves road appearance, and it does these things almost always without additional cost.

The formal study of the esthetics of high-speed-road alinement began in Germany in the 1930s with the work of Fritz Heller, Hans Lorenz (*3*), and others. The German engineers went to considerable trouble and expense to eliminate or modify combinations of vertical and horizontal curvatures that looked awkward when viewed in perspective from a low angle. The following definitive guidelines for achieving a safe and esthetically pleasing three-dimensional alinement are taken from German studies made in recent years.

THREE-DIMENSIONAL ALINEMENT

This section is based on German *Guidelines for the Design of Rural Roads, Part II: Alinement, Section 2: Three Dimensional Alinement* (*RAL-L-2*) (*4*) and the newest knowledge in this field (*5,6*).

By applying perspective methods, the view of the road can be shown in a single drawing. In these guidelines, only the perspective view of the driver is considered. Other perspective views should not be used for the three-dimensional evaluation of the road. For example, a perspective view from a bird's eye may show a sharp curve [Figure 1(*a*)], which in reality is not critical because of driving dynamics or optical deficiencies [Figure 1(*b*) and (*c*)].

The goal of these guidelines is to produce the best alinement that provides optimum safety and traffic quality. Well-balanced, curvilinear road sections in which each single design element contributes to a good road characteristic should be created. Well-balanced sections eliminate unsafe feelings and driver discomfort. With the use of these guidelines, the designer will be able to recognize and evaluate preliminary road designs that result from superimposing selected horizontal and vertical design elements. In this way the designer can create three-dimensional design elements that achieve perceivable and safe road characteristics.

Elements of Three-Dimensional Alinement

Although the design of a road or roadway plans may consist of individual elements, the combination of horizontal alinement

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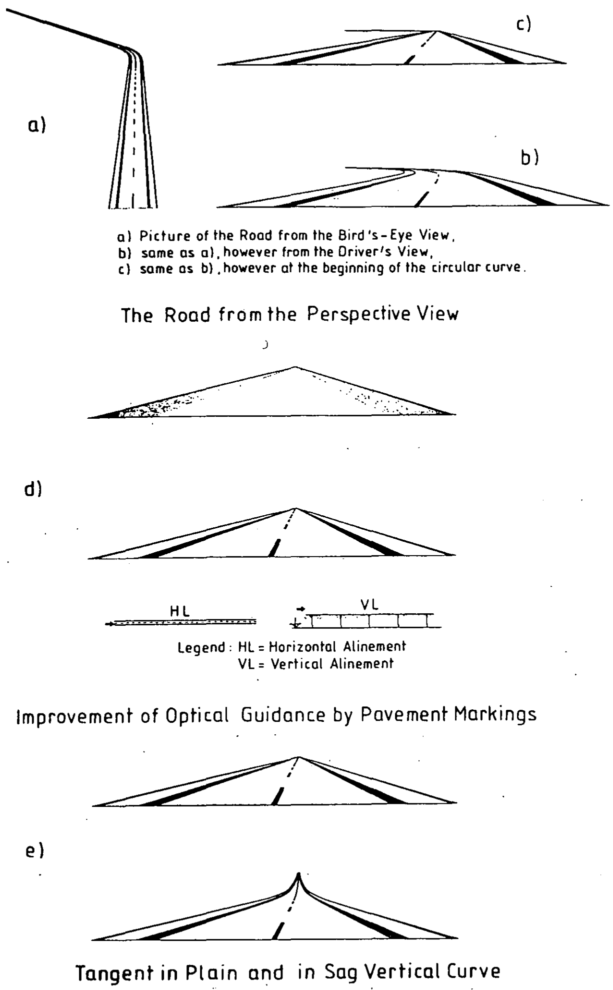


FIGURE 1 Examples of different perspective views (*4*). The road from different perspective views: (*a*) picture of the road from the bird's eye view; (*b*) same as (*a*), however from driver's view; (*c*) same as (*b*), however at the beginning of the circular curve; (*d*) improvement in optical guidance by pavement markings; and (*e*) tangent in plain and in sag vertical curve.

(plan) and the vertical alinement (profile) results in a spatial or three-dimensional creation because of the superimposition of all design elements. The resulting driving space can be described in its sequence with the concept of *road characteristics*. It includes the whole of the structural elements and determines the driving behavior of the motorist. Road characteristics should not be changed greatly over short roadway sections. A consistent sequence of images of the driving space should be balanced in relationship to the design parameters among themselves (relation design). Different roadway sections should be connected by gradual transitions with each other (*5*) (see the paper by Lamm and Smith, this Record).

Thus, in the design of highway alinements, horizontal and vertical design elements are necessarily superimposed. Combining the cross-section of the road, which includes shoulders, pavement width, pavement lane, and edge markings, with horizontal and vertical design elements results in a three-dimensional design element. The design of any road is made up of a series or a sequence of three-dimensional design elements. Typical three-dimensional design elements become important for every new highway geometric design guideline (Figure 2).

Design of Driving Space

The creation of a good view of the road (optical guidance by the roadway) requires a tuned design of the roadway edge (surface guidance) and of the driving space (spatial guidance) with regard to the function of the road. It can be influenced positively by the sensible selection and use of all given possibilities (three-dimensional design elements of the roadway, pavement markings, slopes, embankments, plantings, engineering structures, traffic signing, and directional signing) (*5*).

A good view of the road (optical guidance) is an important issue in conjunction with safety and traffic flow along a roadway section. Usually this can be achieved if the view of the road appears to blend into the surroundings and if the direction of the road is readily apparent.

Furthermore, the optical guidance by the road is created by the perspective view of the road. For example, the direction of the road becomes more obvious as pavement edges and lane lines are marked more distinctly [Figure 1(*d*)]. Pavement markings are of special significance in superelevated sections and where lanes are widened (surface guidance). Detailed information, including numerous examples that show the three-dimensional alinements of highways (spatial guidance), is given in Figures 3 and 4.

Horizontal Design Elements

Tangent

Long tangent sections of highways are monotonous and fatiguing. They can mislead the driver into traveling at excessive speeds and increase the danger from headlight glare at night. Therefore, long tangents with constant grades must be avoided, and the maximum length in the German design guidelines is limited numerically, in meters, to 20 times the design speed in kilometers per hour.

The unfavorable impression caused by long tangents can be reduced by the use of a sag vertical curve with a long length and large radius [Figure 1(*e*)].

Short tangent segments between two horizontal curves in the same direction should be avoided [Figure 3(*a*)]. If such designs cannot be eliminated, it is important that a minimum length be used between the two curves. The minimum length of the tangent segment should correspond to a numerical value, in meters, about four to six times the design speed in kilometers per hour.

Curves

Short circular curves between tangents appear as optical breaks [Figure 3(*b*)] if they are viewed from a long distance. Such an optical break can be avoided [Figure 3(*c*)] by connecting the two tangents with a long horizontal curve (see paper by Lamm and Smith, this Record).

Vertical Design Elements

Tangent Segments

A short tangent used between two succeeding sag vertical curves can give the impression of a crest vertical curve [Figure 3(*d*)] and

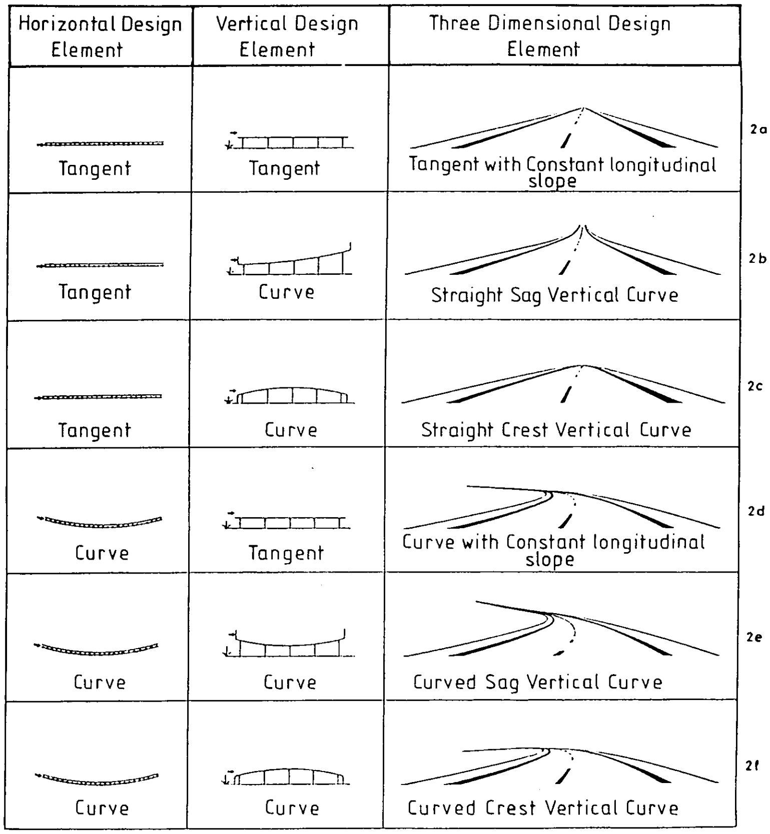


FIGURE 2 Three-dimensional design elements originated by superimposing tangents and circular curves (*4*).

should be avoided. A better solution, a long sag vertical curve, is shown in Figure 3(*e*). The same is true for a short tangent between two succeeding crest vertical curves. Such a design may give the impression of a sag vertical curve [Figure 3(*f*)] and should be avoided. A better solution, a long crest vertical curve, is shown in Figure 3(*g*). In addition, the greater the distance a motorist can see ahead on the road, the longer a sag vertical curve should be to eliminate visual breaks.

Sag Vertical Curves

The sag vertical curve is the three-dimensional design element with the best visual qualities and optical guidance [Figure 1(*e*)]. However, there is one exception: the use of a short sag vertical curve between long sections with constant grades should be avoided. In this case it does not matter whether the horizontal alinement is on a tangent section or a curve [Figures 4(*a*) and 4(*b*), respectively]. In both cases, a visual break in the perspective view occurs. The length of sag vertical curves on embankments usually can be increased considerably without a large increase in earthwork costs.

Crest Vertical Curves

The crest vertical curve represents the most critical design element when considering good visual qualities. The influence of a crest vertical curve is especially critical with short lengths that cause insufficient sight distances. Crest vertical curves with minimum stopping sight distances should be avoided on the mainline roadway if at all possible. The main consideration in using longer lengths is earthwork costs. With the availability of user-friendly earthwork programs and the perspective plot capabilities of the programs, it is now an easy task to design and test many alternative profiles.

Consequences

On mainline roadway sections, avoid visual breaks that are the result of short horizontal and vertical curves or their combination. Instead, strive to use generous design elements. Short curves lead to inconsistencies at the roadway's edge. Compare Figures 3(*a*) and 3(*b*) with Figure 3(*c*) and Figures 3(*d*) and 3(*f*) with Figures 3(*e*) and 3(*g*). Figures 4(*a*) and (*c*) also show designs that should be avoided.

Furthermore, the following should be avoided:

- Diving: the partial disappearance of the road from the driver's view with reappearance in the extension of the just-passed roadway section [Figure 4(*d*)].
- Jumping: similar to diving but with displaced reappearance [Figure 4(*e*)].
- Fluttering: multiple diving or a rapidly rolling profile [Figure 4(*f*)].

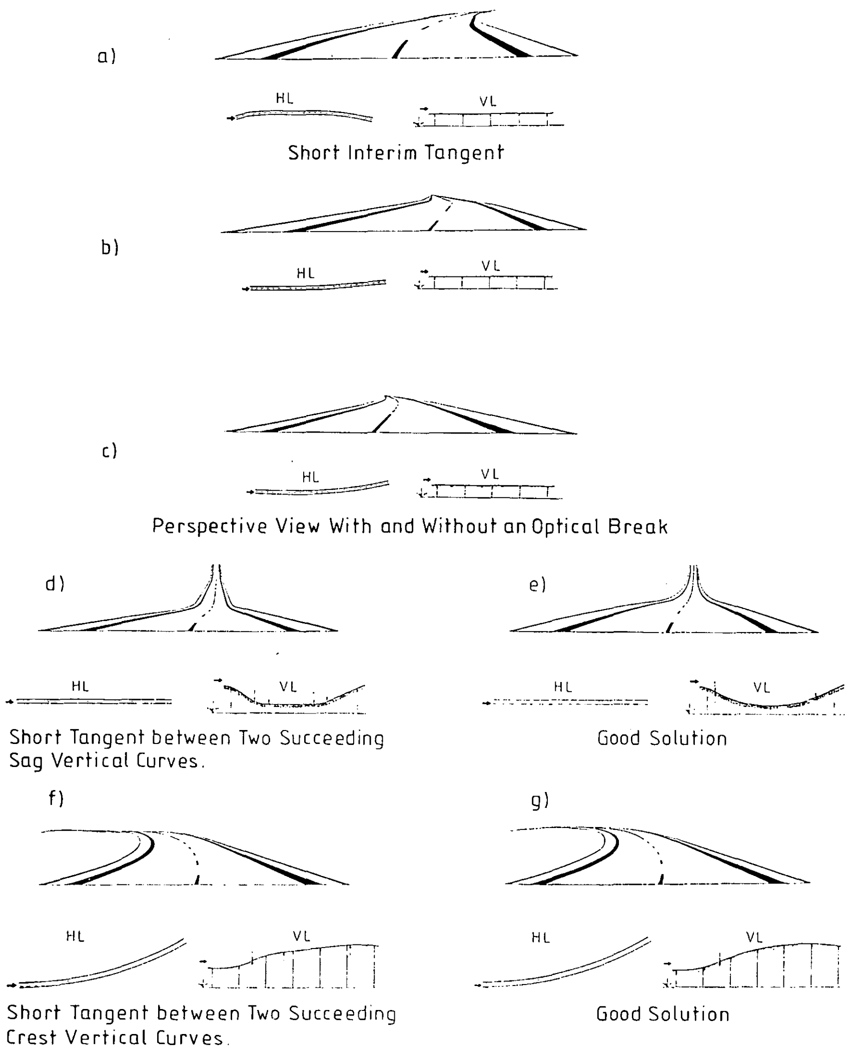


FIGURE 3 Examples of poor and good solutions in Germany (*4*): (*a*) short interim tangent; (*b*) and (*c*) perspective view with and without an optical break; (*d*) short tangent between two succeeding sag vertical curves and (*e*) good solution; (*f*) short tangent between two succeeding crest vertical curves and (*g*) good solution.

- Broken-back vertical curve: a short tangent section between two sag vertical curves [Figure 3(*d*)].

All these designs may lead to critical driving maneuvers especially because of visual misconceptions.

SEQUENCE OF DESIGN ELEMENTS AND SUPERIMPOSITION OF ELEMENTS

Horizontal Alinement

The safety of a motorist is not potentially impaired by the use of a series of smaller-radius curves for a winding alinement. Despite the sharp curvature, a more or less curvilinear alinement does exist [Figure 5(*a*)]. However isolated sharp curves in the course of a gentle alinement are noncurvilinear, can be considered dangerous, and should be avoided [Figure 5(*b*)]. Reliable margins for the range of radii of curve sequences that ensure satisfactory operational practices are given by the authors in another paper, this Record. The same concept can be applied to sight distance problems along the course of a road.

Vertical Alinement

Note that in U.S. practice vertical curves are portions of vertical parabolas and the length (L) of the vertical curve is usually specified in the design.

In German practice the radius (R) of a circular arc is usually selected or specified for vertical curve design. Note, however, that the Germans replace the circular vertical curve with a vertical parabola for ease in computing elevations along the vertical curve.

It can be shown that a circular arc can be used to replace a parabola (and vice versa) as the typical highway vertical curve with almost no error. In the United States the rate of vertical

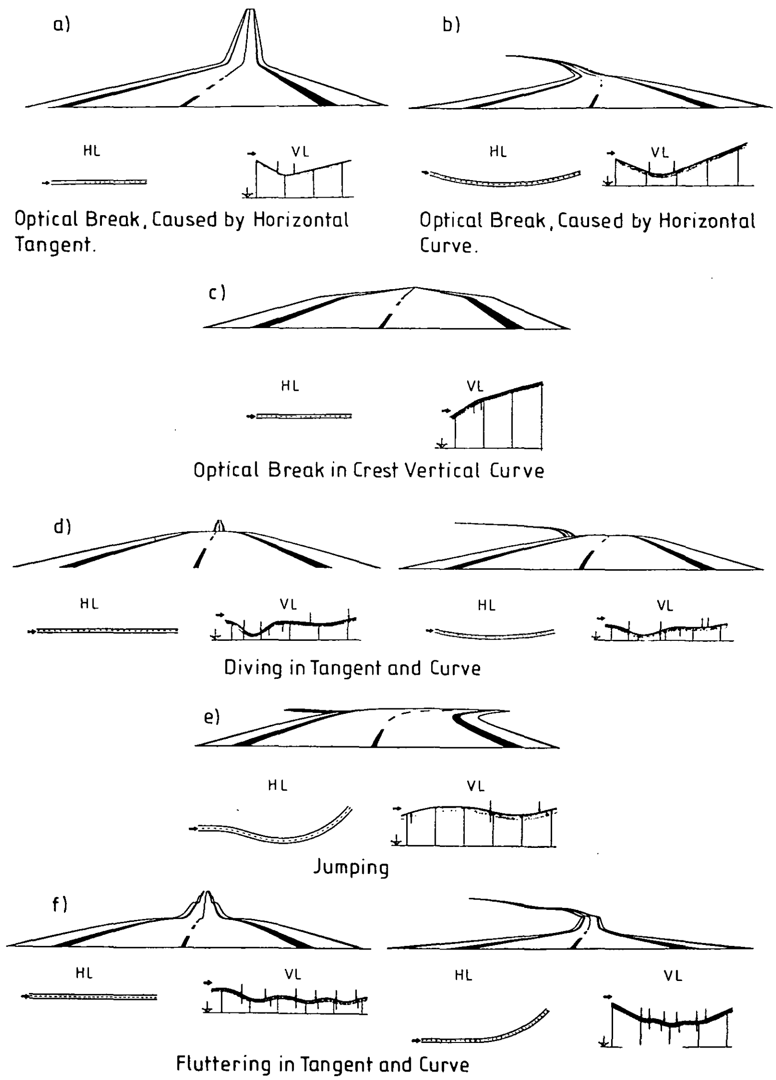


FIGURE 4 Design cases to be avoided (*4*): (*a*) optical break caused by horizontal tangent; (*b*) optical break caused by horizontal curve; (*c*) optical break in crest vertical curve; (*d*) diving in tangent and curve; (*e*) jumping; (*f*) fluttering in tangent and curve.

curvature (K) is the length (in feet) of vertical curve (a portion of a vertical parabola) per percent change of grade (A). The length of vertical curve is, therefore, written $L = KA$ (*2*).

It can be shown that the radius of an equivalent vertical parabolic curve is very close to $R = 100K$ (feet) or $R = 30\ K$ (meters) for typical highway designs.

To provide stopping sight distance (U.S. criteria) for a design speed of 110 km/hr (70 mph), the minimum recommended K values range from 220 for sags to 540 for crests (*2*). The minimum recommended radii would then range from 6700 m (22,000 ft) for sags to 16 200 m (54,000 ft) for crests.

For sequences of design elements in the vertical plane, the following must be considered (Figure 5).

1. In hilly topography, the radii of crest vertical curves should be larger than the radii of sag vertical curves. This concept provides a longer sight distance for the crest vertical curve [Figure 5(*c*)]. Significantly longer sight distances provide a greater feeling of safety to the driver.

2. For smaller differences in the elevation of a roadway (up to 10 m) and on a roadway with a flat topography, the radii of sag vertical curves should be larger than those of crest vertical curves. This concept takes into consideration that a motorist can see the road for a longer distance in flat terrain and therefore provides the motorist with a smoother, more visually satisfying view of the course of the road [Figure 5(*d*)].

3. Quick sequences of short crest and sag vertical curves should be avoided.

Superimposition of Elements

With respect to the superimposition of horizontal and vertical alinements, the ratio between radii of horizontal curves (R) and radii of sag vertical curves (H_w) cannot be selected arbitrarily but

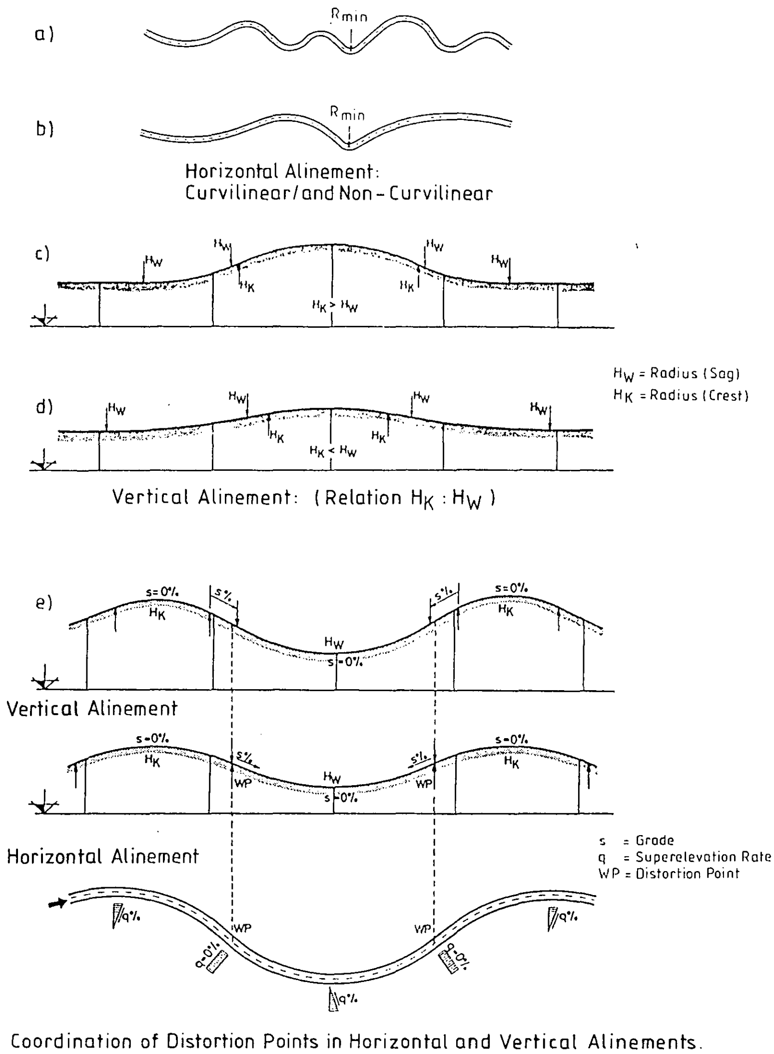


FIGURE 5 Element sequences and superimposition (Germany): (*a*) and (*b*) horizontal alinement: curvilinear and noncurvilinear; (*c*) and (*d*) vertical alinement (relation H_K:H_W); (*e*) vertical alinement, horizontal alinement, and coordination of distortion points in horizontal and vertical alinements.

must be related or tuned to each other. To achieve a satisfactory three-dimensional solution, experience shows that the ratio R/H_w should be as small as possible. The ratio should be in the range of 1/5 to 1/10.

If these values are exceeded, a perspective analysis of the roadway section is recommended. This can be accomplished easily by using modern computer systems and user-friendly earthwork programs now available. Such programs are usually part of a computer-aided drawing and design (CADD) system and also involve the use of perspective plot programs.

In flat topography, the radii of crest and sag vertical curves should be longer than the radii of horizontal curves. Furthermore, a favorable view of the road is normally guaranteed if the distortion (revolving) points of both the horizontal and vertical alinements are set to approximately coincide. This is shown in Figure 5(*e*). This can be accomplished if the curves in both the horizontal and vertical alinements are placed at approximately the same location and have about the same length. In this way, the distortion points then lie at about the same spot and a sufficient longitudinal slope for drainage is guaranteed at the zero points of the superelevation rate.

Therefore, with such coordination the number of distortion points in the horizontal and vertical planes should always be the same. In addition, by designing areas with low superelevation rates in this way, sufficient longitudinal grades for drainage are attained, and in areas with low longitudinal grades sufficient superelevation is available.

In hilly or mountainous topography with steeper longitudinal grades, it may be desirable to select a segment of constant grade between the ends of consecutive crest and sag vertical curves [see upper vertical alinement of Figure 5(*e*)]. In this case, the distortion point of the horizontal alinement should be set nearer the beginning of a sag vertical curve. This type of design then enables the driver to recognize, in advance, the distortion point of the horizontal alinement.

If the local topography does not allow the designer to fit together the distortion points as shown in Figure 5, it then becomes desirable for safety reasons to allow drivers to perceive the exact course of the roadway as soon as possible. Because the largest part of the road is perceived visually, it is necessary to design the visual image on the basis of the driver's perspective. This can best be accomplished by designing horizontal curves to lead crest vertical curves. Designing a highway in this way allows motorists to perceive where the highway is going before they get to the curve (*5*). For example, the views of the road shown in Figure 4(*d*) and 4(*e*) should be avoided.

U.S. RECOMMENDED PRACTICE

In this section some currently recommended practices in the United States are presented and are related to the practices recommended in Germany described above. In some cases the U.S. literature may not properly credit the German sources.

The following are some guiding rules for the satisfactory three-dimensional appearance of highway alinement. They are taken from *Practical Highway Esthetics* (*1*), The 1990 Green Book (*2*), Pushkarev (*7*), and Cron (*8*).

• Curvature in the horizontal plane should be accompanied by comparable curvature in the vertical plane, and vice versa (*1,7*). Thus, the grade line for a long flat horizontal curve should be smooth and flowing and not interrupted by short dips and humps. Figure 6(*a*) shows an unpleasant view, and Figure 6(*b*) shows a more pleasing view.

Comment: The earlier discussion of vertical design elements (sags) [Figures 4(*a*) and (*b*)] and consequences [Figure 4(*f*)] show what can happen if this recommended practice is not followed.

• Awkward combinations of curves and tangents in both the horizontal and vertical planes should be avoided (*2*, item 4, p. 294). The most prominent of these combinations is the broken-back gradeline, that is, two sag curves in the same direction connected by a short tangent [Figure 6(*c*)]. Vertical broken backs are visibly prominent only when short vertical curves are used. The broken-back appearance can be corrected by using longer vertical curves at each end of the short grade tangent or by eliminating the short grade tangent. The remedy for horizontal broken-back curves is to replace the tangent with a flat curve or to use at least 500 m (1,500 ft) of tangent section between the two horizontal curves in the same direction.

Comment: The earlier discussion of vertical design elements (tangents) and consequences [compare Figure 3(*d*) and 3(*e*)] address this issue and agree.

• Horizontal and vertical curvatures should be coordinated to avoid combinations that appear awkward when viewed from a low angle (*1,2,7,8*). Ideally, the vertices of horizontal and vertical curves should coincide [Figure 6(*d*)]. This statement corresponds to the earlier discussion of the superimposition of elements [Figure 5(*e*)]; however, this is not always possible. A reasonably satisfactory appearance will result, however, if the vertices of the horizontal and vertical curves are kept apart by not more than one-quarter phase. Skipping a phase in the plan while keeping the profile vertices in phase will result in reasonably good coordination and appearance [Figure 6(*e*)]. A shift of one-half phase will result in poor coordination and appearance, as shown in Figure 6(*f*).

Changes in horizontal and vertical curvature should not occur at or near the same point. When combining horizontal and vertical curves, the former should be somewhat longer than the latter, and where the vertices of the curves are slightly out of phase, the vertical curve should lie completely within the horizontal curve.

According to Rose (*1*, p. 59,60) the coordination of horizontal and vertical alinements is seldom a serious problem when the radius of a horizontal curve is 1800 m (6,000 ft) or longer and when vertical curves are longer than about 128 m (420 ft) for each percentage of gradient change. Alinements with sharper curvatures than this should be studied for their three-dimensional appearances with computer-drawn perspective plots.

Comments: note that the 128 m (420 ft) per percent grade change is the K value, that is, the rate of vertical curvature [length (in feet) per percent of grade change (A)]. As noted earlier, this would translate into a minimum vertical curve radius of 42,000 ft ($R = 100K$) or 12 600 m ($R = 30K$, in meters).

The ratio of the horizontal curve radius to the vertical curve radius is 1800/12 800 m (6,000/42,000 ft), or 1/7, and is within the 1/5 to 1/10 minimum ratio recommended in Section 3.2.

• In the Green Book (*2*, item 3, p. 296) the following is noted:

> Sharp horizontal curvature should not be introduced at or near the top of a pronounced crest vertical curve. This condition is undesirable in that the driver cannot perceive the horizontal change in alinement, especially at night when the headlight beams go straight ahead into space. The difficulty of this arrangement is avoided if the horizontal curvature leads the vertical curvature, i.e., the horizontal curve is made longer than the vertical curve. A suitable design can also be achieved by using design values well above the minimums for the design speed.

Comment: This agrees with the earlier discussion on crests and the superimposition of elements. See Figure 4(*c*) to (*e*), which show designs that are to be avoided.

• The Green Book (*2*, item 4, p. 296) continues:

> Somewhat allied to the above, sharp horizontal curvature should not be introduced at or near the low point of a pronounced sag vertical curve. Because the road ahead is foreshortened, anything but flat horizontal curvature gives an undesirable distorted appearance. Further, vehicular speeds, particularly of trucks, often are high at the bottom of grades, and erratic operation may result, especially at night.

Comment: This statement addresses the same concern as described in the section An Interesting Phenomenon. The authors suggest the following study: Using a CADD system, prepare a series of perspective drawings from the driver's viewpoint. The objective should be to determine if sag vertical curves superimposed with horizontal curves result in perspective views that make the horizontal curve appear flatter than it is in reality.

A properly designed study, as hypothesized in the section An Interesting Phenomenon, could likely determine the visual effects of varying the lengths of and overlapping the curves. If the hypothesis is true, then the situation is one in which a driver's expectancy is violated. This could lead to improper actions by drivers and, possibly, increased safety problems.

• The length of highway that can be seen at one time by the motorist should be limited, but adequate sight distance should be preserved (*1*). There should not be more than two course changes in horizontal alinement [Figure 7(*a*)] or three breaks in the vertical grade line in the view of a driver at any point [Figure 7(*b*)]. In particular, a disjointed appearance should be avoided. This may

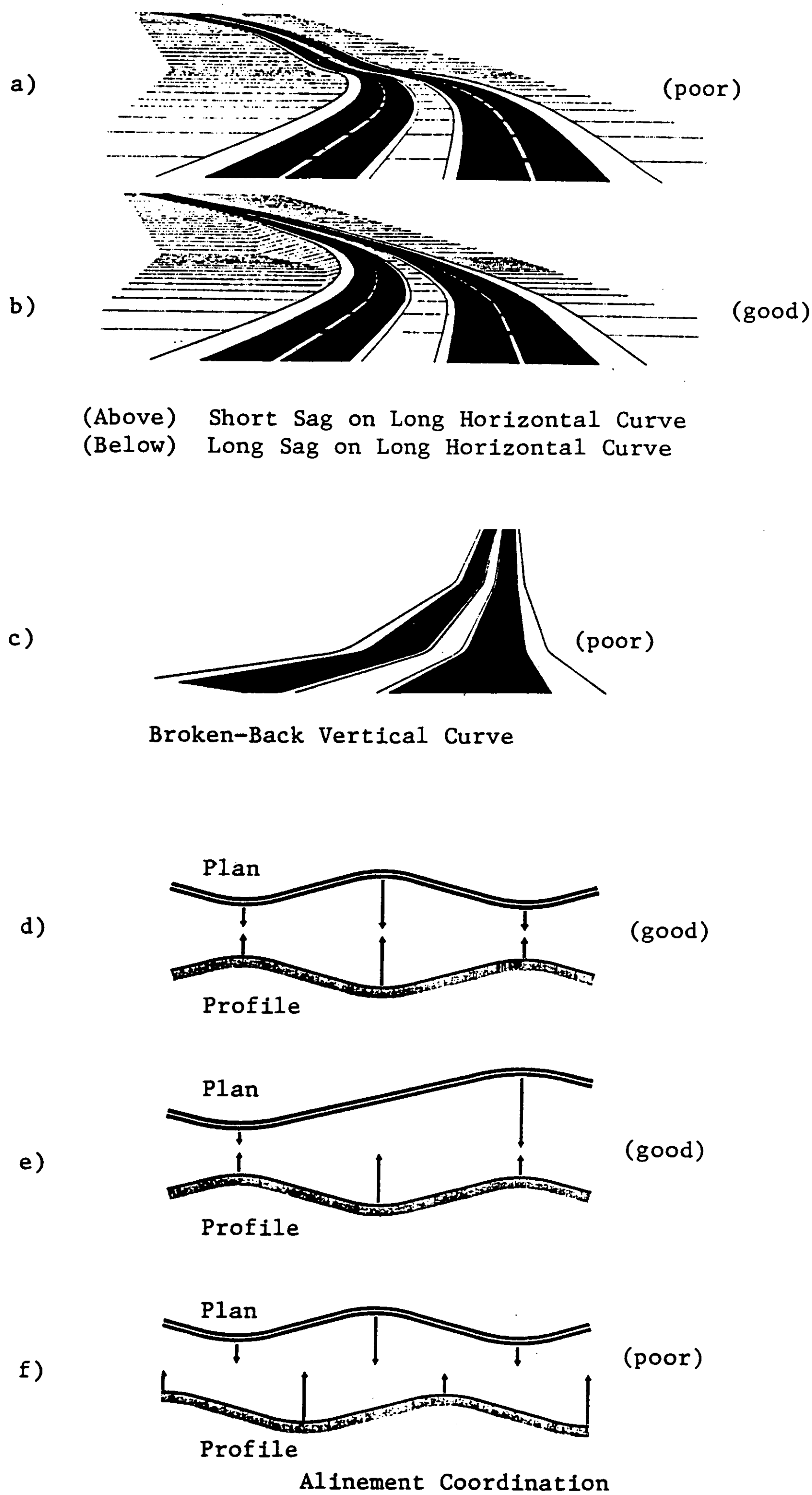
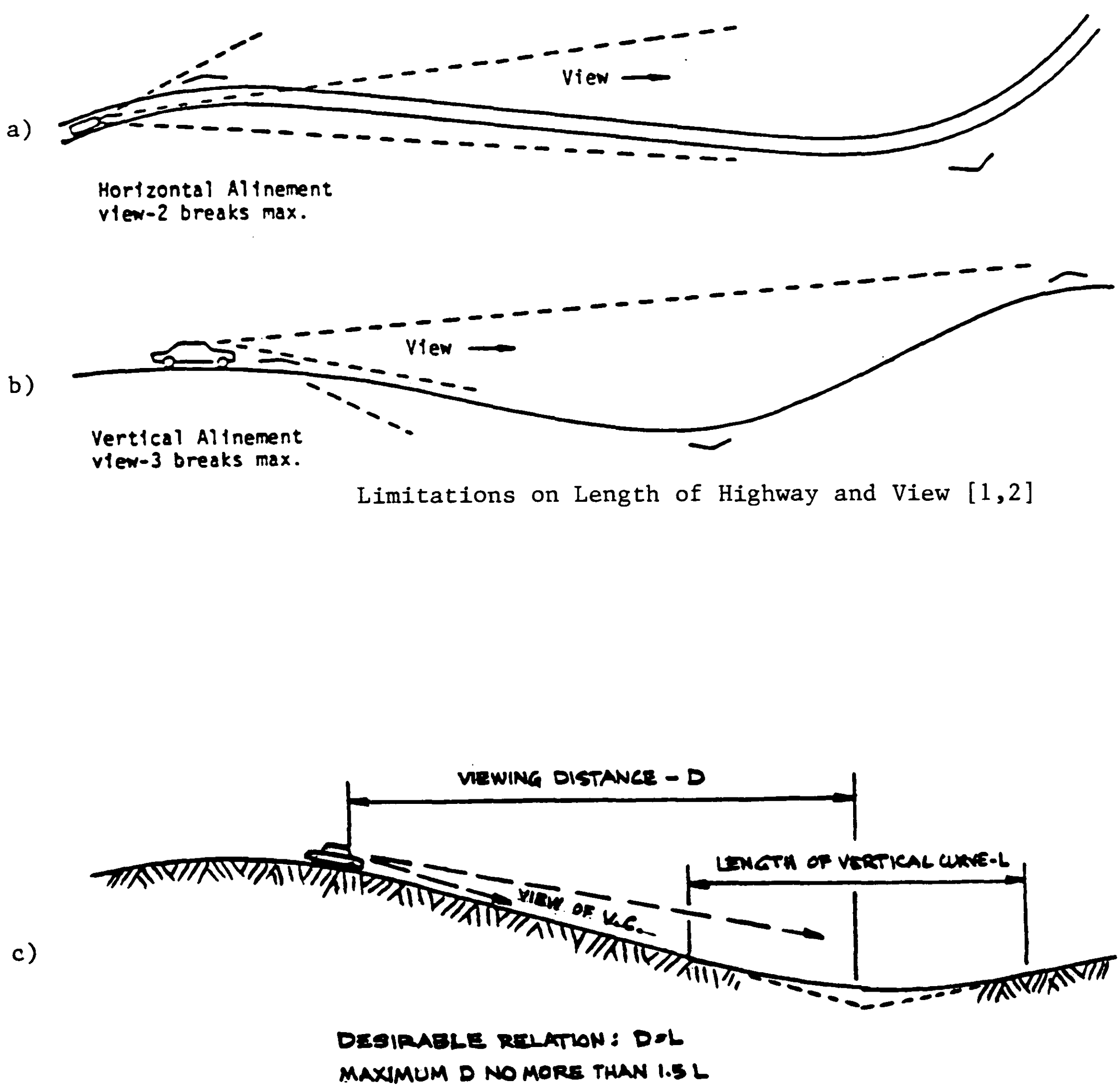


FIGURE 6 Examples of poor and good solutions, United States (*7*). (*a*) Short sag on long horizontal curve (poor); (*b*) long sag on long horizontal curve (good); (*c*) broken back vertical curve (poor); (*d*) alinement coordination (good); (*e*) alinement coordination (poor); (*f*) alinement coordination (good).



Viewing Distance vs. Length of Vertical Curve [1]

FIGURE 7 Fundamental issues for optical (visual) guidance, United States: (*a*) horizontal alinement, view: two breaks maximum; (*b*) vertical alinement, view: three breaks maximum; limitations on length of highway and view (*1,2*); (*c*) viewing distance versus length of vertical curve (V.C.) (*1*).

occur when the beginning of a horizontal curve is hidden from the driver by an intervening summit while the continuation of the curve is visible in the distance beyond (*1,2*) [Figures 4(*d*) and 4(*e*)]. It may also occur when long tangents are laid in rolling terrain such that the road appears as a series of segments of diminishing size as it passes over successive hilltops ahead [Figure 4(*f*)].

• For satisfactory appearance, curves, both horizontal and vertical, should usually be considerably longer than the minimum design standards, on the basis of safety and operational ease, would require (*1,7*). This is particularly true of sag vertical curves, which appear to the driver as sharp angles or kinks when seen from a distance. To avoid the appearance of a kink, the length of a sag vertical curve should be about the same as the viewing distance from which the curve is first perceived by the driver or, as a minimum, at least 0.6 of the viewing distance [Figure 7(*c*)]. The length of horizontal curves (in feet) should be at least 15 times the design speed of the highway (in miles per hour) and preferably twice that. [In Standard International units, the length of the curve (in meters) should be at least 3 times the design speed (in kilometers per hour) and preferably twice that length.]

Comment: The earlier discussion of vertical alinement in the section Sequence of Design Elements and Superimposition of Elements appears to support this viewpoint.

- Pushkarev (*7*) rightly claims that crest vertical curves do not pose the esthetic problems that sag curves do if they are so high that the road terminates visually on the horizon line near the crest. He makes an interesting observation that the minimum stopping sight distances [625 to 850 ft at 70 mph (*2*)] beyond which the driver does not see while going over the crest "visually often appears quite precarious, even though it is functionally safe." He implies that (a) much larger radius crest curves (generally longer crest curves) should increase the driver's feeling of safety and security and (b) a driver's feeling of safety derived from the view of the road is very important.

Comment: This agrees with the earlier discussion in the section Sequence of Design Elements and Superimposition of Elements. Perhaps someday the following will be quantifiable: (a) how to provide the driver with a feeling of safety and security and (b) the importance, in terms of safety, of such driver feelings.

SAFETY ASPECTS

General Statements

Two important considerations in highway design are design speed and sight distance. These in turn vary depending on the horizontal and vertical alinement characteristics of the highway. Favorable horizontal and vertical alinements allow for higher design speeds and extended sight distances and are expected to generally result in increased motorist safety. However, little information exists on the effects of vertical alinement on accidents. Also, although cer-

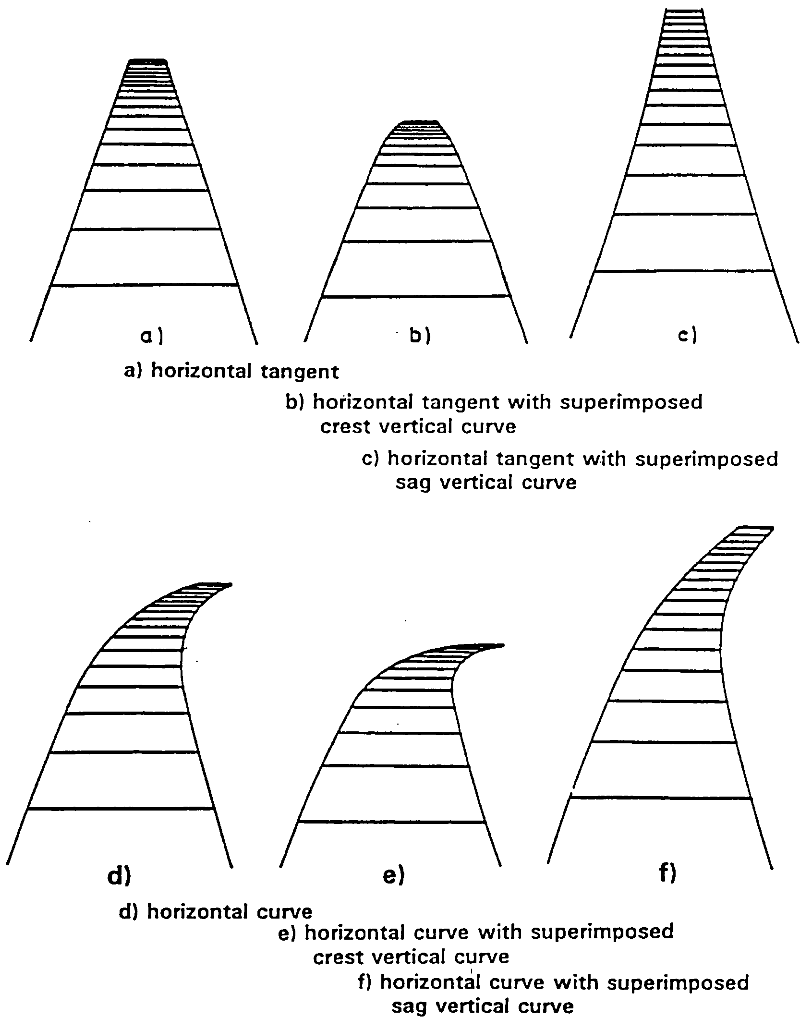


FIGURE 8 Different three-dimensional views by superimposing vertical and horizontal curves (*9*): (*a*) horizontal tangent; (*b*) horizontal tangent with superimposed crest vertical curve; (*c*) horizontal tangent with superimposed sag vertical curve; (*d*) horizontal curve; (*e*) horizontal curve with superimposed crest vertical curve; (*f*) horizontal curve with superimposed sag vertical curve.

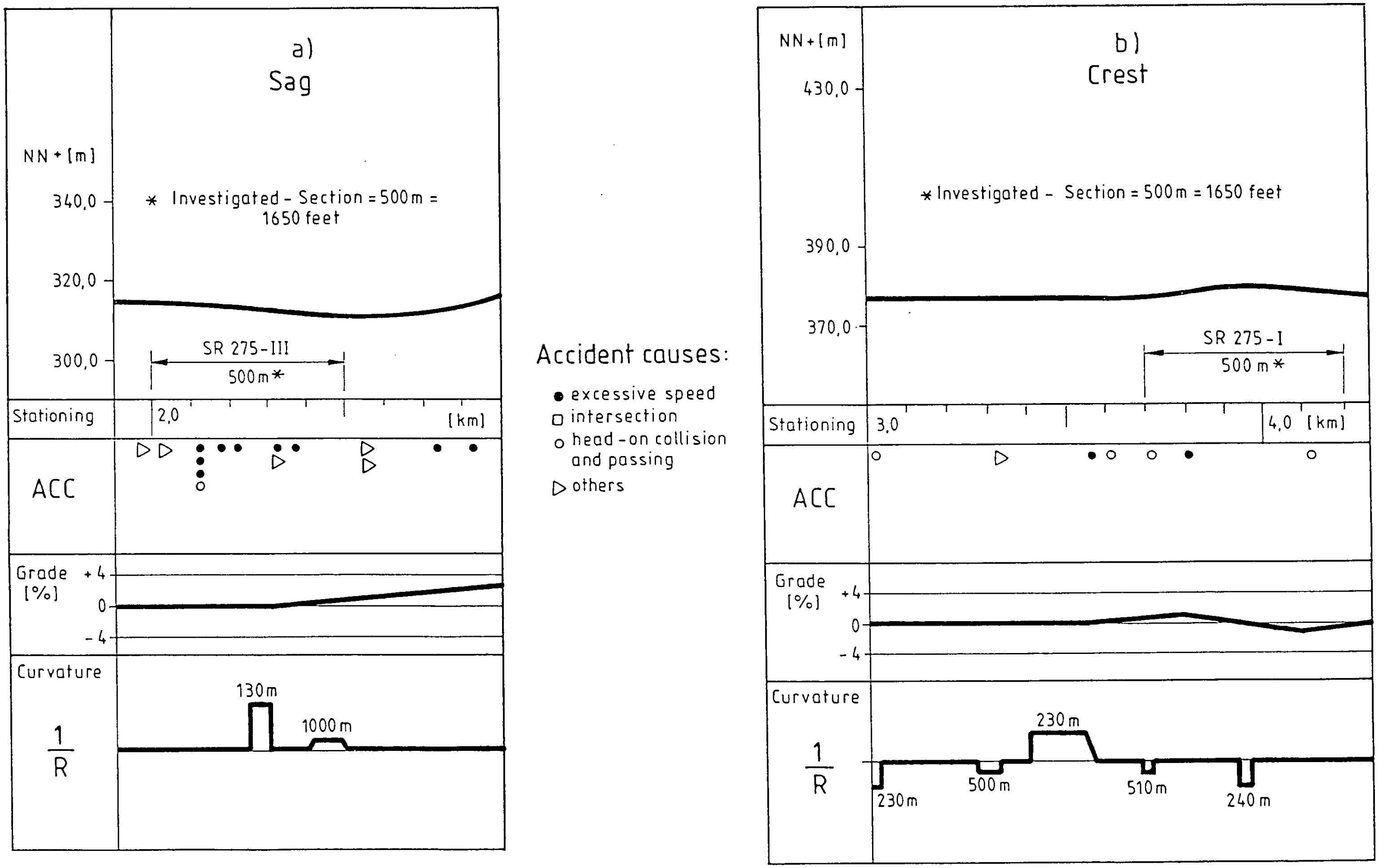


FIGURE 9 **Influence on the accident (ACC) situation by the superimposition of horizontal and vertical curves: (*a*) sag and (*b*) crest.**

tain combinations of unfavorable horizontal and vertical alinements are expected to increase accident frequency and severity, not much about the specific combinations of such high-accident alinements is known.

A safety evaluation process for highway geometric design was developed especially for the 1993 Transportation Research Board Conference Session Cross-Section and Alinement Design Issues and is discussed in the papers by Lamm and Smith and Lamm, et al. this Record.

Additionally, the impacts of individual design elements (like radius of curve, lane width, grade, and sight distance and their interactions on the safety aspects of a highway were analyzed in the paper by Choueiri et al., this Record. With regard to gradients, it was found for two-lane rural highways (on the basis of the present international study) that (a) grades under 6 percent have relatively little effect on the accident rate, and (b) a sharp increase in the accident rate was noted on grades of more than 6 percent.

An Interesting Phenomenon

The problems of visual perception when horizontal and vertical curves are superimposed was recognized very early in Germany (*9*). Figure 8 shows how sag and crest vertical curves change the three-dimensional view of roadways when they are superimposed with horizontal curves.

It is especially interesting to see how a vertical curve may change the perception of the curvature of a horizontal curve [Figure 8(*e*) and (*f*)]. This leads to the hypothesis that sag vertical curves superimposed with horizontal curves may result in perspective views that make the horizontal curve appear flatter than it is in reality. Therefore, in those situations higher operating speeds that could cause higher accident risks might be expected. This would be in comparison with those in horizontal curves without any superimposition of sag vertical curves. A superimposition of a crest vertical curve with a horizontal curve should reveal opposite results [Figure 8(*e*)] (*9,10*). A study of drivers' perceptions of such curve combinations by using a CADD system was recommended earlier (*2*, item 4, p. 296).

Figure 9 (*10*) shows the geometries, accident locations, and accident causes for a typical horizontal and sag vertical curve superimposition and for a horizontal and crest vertical curve superimposition. The accident rate at the sag vertical curve [Figure 9(*a*)] was 8.3 accidents per 10^6 vehicle km, in comparison with the average accident rate of 3.6 accidents per 10^6 vehicle km over the entire lengths of the observed state routes. Excessive speed was the single most frequent cause of accidents at these sites. The sag vertical curve site was especially hazardous. The 10 accidents within section SR 275-III resulted in three fatalities and three injuries. The other two sag vertical curve sites also revealed very critical results. In contrast, at the crest curve site depicted in Figure 9(*b*), there were only three accidents during the same time period. The accident rate was 1.4 accidents per 10^6 vehicle km, in comparison with an average rate of 3.6 accidents per 10^6 vehicle km. The results support the hypothesis (even if it is not statistically valid) that superimposed sag vertical curves may create a potential hazard by making the horizontal curve appear flatter than it really is.

The mathematical background and further discussion of this phenomenon can be found in an article by Osterloh (*9*). It is based mainly on human factor-related errors.

RECOMMENDATIONS

It is recommended that the following be specifically considered for inclusion in the Green Book (*2*):

1. Establishment of appropriate horizontal curvilinear alinements as described in a paper by Lamm and Smith, this Record with maximum longitudinal grades of up to 5 percent (with exceptions of 6 percent).
2. Description of the important three-dimensional design elements.
3. Increase in the use of perspective plots, which can help to detect numerous visually poor designs. This can be accomplished by using a CADD system now available in most engineering offices.
4. A brief discussion on the relationship of the rate of vertical curvature, *K*, to an equivalent vertical curve radius, *R*: R (ft) = $100K$ (feet/percent grade change) or $R = 30K$, where *R* is in meters.
5. Rules for ratios of horizontal to vertical curve radii (1/5 to 1/10).
6. Hilly topography, $R_{crest} > R_{sag}$; flat topography, $R_{sag} > R_{crest}$.
7. Rules for the coordination of distortion points.
8. Rules for limitations on the length of highway motorists can see.
9. Emphasizing the safety benefits of alinement coordination.

CONCLUSION

Both this paper and its companion by Lamm and Smith, this Record, present numerous indirect visual and safety-related issues. Safety aspects are especially important in the case of curvilinear alinements with grades of from 5 to 6 percent. The two papers are intended to be used together by highway designers.

The common purpose of these papers is to assist designers in avoiding horizontal and vertical designs that, in subtle negative ways, may diminish the driver's feeling of comfort, certainty, and safety and that at times may violate the driver's expectations. The proposed design guidelines should help to create highways that are much freer from operating speed inconsistencies and should reduce high-risk driving maneuvers because of errors in driver judgment. Modern geometric highway designs that correspond to the guidelines outlined in both papers would thus include more safety-related issues. However, it should be noted that some of the safety assumptions are evaluated only in a qualitative manner.

Overall, it can be concluded that many recommendations and rules for achieving good visual three-dimensional alinements (mostly on the basis of practical experience) exist. However, sound quantifiable design criteria with special emphasis on traffic safety could not be found, especially for highways where grades are greater than 6 percent. No accident analysis research work has been conducted for such grades either in Europe or in the United States. Three-dimensional alinement, a very complex component in the highway geometric design process, still represents the weakest link in the overall design of highways.

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Minimum Horizontal Curve Radius as Function of Grade Incurred by Vehicle Motion in Driving Mode

MARIOS KONTARATOS, BASIL PSARIANOS, AND APOSTOLOS YOTIS

An enriched "bicycle" model was developed to describe the driving mode during the motion of a passenger car. The bicycle model is contrary to the mass-point model, which is suitable for the examination of the braking mode of a passenger car. The analysis concludes that there is a strong relationship between the radius of the horizontal curve and grade, which is found by studying vehicle motion on a helical surface. In some cases values of the minimum radius of the horizontal curve derived from the relationship exceed those suggested by AASHTO (1990) or RAS (1984). This means that, in such cases, existing guidelines lead to underdesigned values, because they do not consider that the driving mode during vehicle motion is critical.

In modern road design theory, the determination of the minimum horizontal curve radius for a certain speed is carried out on the basis of the following assumptions:

1. The vehicle is reduced to a simple mass-point,
2. The configuration of the road is a plane curved surface (i.e., there is no grade), and
3. The motion of the vehicle is governed by side friction values recommended by design guidelines.

Although the first two assumptions represent globally accepted design practices, two distinctive approaches exist in the third assumption. According to the first approach accepted by AASHTO policy in 1990 (*1*) (AASHTO-1990), side friction values are established on the basis of the comfort of the driver while negotiating a curve. This actually means that curve design is directly related to the dynamic constraints imposed on the vehicle as it moves in the driving mode. On the contrary, the second approach, which is accepted, among others, by the German 1984 RAS policies (*2*) (RAS-1984), refers to the motion of the vehicle under braking conditions. Specifically, according to the latter approach, the maximum side friction values that are used are such that considerable reserves of friction are disposed at the longitudinal direction in the case of braking. Although this difference in approach is implied in various guidelines, accidentally similar factors for limiting the side friction values were established.

The adoption of these assumptions in current road design theory, although validated by empirical data, leads to the calculation of minimum radius (R_{MIN}) values as an element independent from other design elements that coexist at the same road segment or other design constraints as vehicle characteristics. An optimal design process, however, can be accomplished by synthesizing and quantifying the interactive relations that exist between design goals, design constraints, and design elements, as Glennon and Harwood (*3*) have clearly pointed out. A model stronger in its ability to describe the cornering motion can be established, giving reliable design criteria for horizontal curves.

This paper intends to contribute to the enhancement of the design modeling tool so that these interactions between design elements can be revealed. Thus a vehicle-road model is formulated in which road geometry is almost exact and the vehicle (passenger car) is defined as a rigid body moving in the driving mode complying with the AASHTO-1990 motion mode. This model is used to determine the value of the minimum horizontal curve radius. Thus, a full extension of all three classical assumptions of the vehicle-road model as mentioned above is accomplished. Finally, an enriched "bicycle" model is developed. The model describes a vehicle that has height, with the forces acting on the interior and exterior wheels being equal, whereas those acting on the front and rear wheels depend on the type of vehicle drive.

Attempts to extend the classical vehicle-road model by some of the three-dimensional road parameters and other operational features of the vehicle are found in the literature (*4–6*). Some of the conclusions reached, however, must be regarded with caution. For example, the finding that the combined effect of grade and cross-slope has no substantial influence on the value of the minimum horizontal curve radius was derived by assuming values of maximum sliding coefficient of friction of between 0.3 and 0.5, which are too high according to established road safety criteria (*4*). Furthermore, because the specific analysis was limited to the braking mode, the calculated values of R_{MIN} were not significantly different from those given by the mass-point model.

In the analysis of the extended vehicle-road model introduced in this paper, the numerical values of parameters used are those accepted by current road and automobile design policies or standards. It is pointed out, however, that no claim of completeness in the overall numerical analysis can be made. Before quantitative statements find their place in design policies, two efforts must be successfully accomplished. First, representative values of the parameters introduced in the present vehicle-road model must be estimated to fit best the prevailing local conditions (e.g., maximum coefficient of friction and representative values of vehicle characteristics). Second, a comparison must be made between the two vehicle operating modes (the braking and driving modes) to establish which one is critical in each combination of horizontal and vertical road geometry elements.

Finally, it should be pointed out that the human factor may impose additional restrictions on the maximum reserve of friction that can be used in the lateral direction (*7,8*). Therefore, the study

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of the complete driver-vehicle-road system may lead to more unfavorable values of various road features (e.g., higher values of R_{MIN}) in comparison with those determined by the analysis presented in this paper.

DETERMINATION OF THE MINIMUM RADIUS

For a given design speed, the minimum curve radius R_{MIN} represents a crucial value for the design of the horizontal alignment. In this paper this value is calculated by the model developed in Appendix B:

$$R_{MIN} = \frac{V^2}{g(n_F f_{Y,MAX} + q)} \tag{1a}$$

$$R_{MIN} = \frac{V^2}{g(n_R f_{Y,MAX} + q)} \tag{1b}$$

for front and rear wheel drive vehicle, respectively, where $1 - f_{Y,MAX} \cdot q \simeq 1$, and the factors n_F and n_R are equal to

$$n_F = \sqrt{1 - \frac{f_{X,F}^2}{f_{X,MAX}^2}} \cdot \left[1 - \frac{h}{l_R} \cdot s - \frac{A_z}{m \cdot g} - \frac{A_x}{m \cdot g} \cdot \frac{h}{l_R}\right] \tag{2}$$

$$n_R = \sqrt{1 - \frac{f_{X,R}^2}{f_{X,MAX}^2}} \cdot \left[1 + \frac{h}{l_F} \cdot s - \frac{A_z}{m \cdot g} + \frac{A_x}{m \cdot g} \cdot \frac{h}{l_F}\right] \tag{3}$$

In the above expressions, $f_{X,MAX}$, which is a function of speed, is the maximum available tangential coefficient of friction. (The abbreviations for the other parameters are defined after Appendix B in the section Nomenclature.) To make use of Equations 1a and 1b, the value of $f_{X,MAX}$ must be greater than the sliding coefficient of friction $f_{X,SL}$ suggested by the current design policies. This is because the present model refers to the driving and not the sliding mode of vehicle movement. The factor by which this should be increased, however, is open to further discussion. Because the purpose of this paper is to introduce the unfavorable safety conditions that may arise in the selection of R_{MIN} if only the driving mode is considered, the peak value of friction coefficient as a function of slip $f_{X,MAX}$ was chosen (*9,10*). This value, being 10 to 40 percent higher than the sliding coefficient of friction, was selected here to remain constant at 30 percent, that is,

$$f_{X,MAX} = 1.3 \cdot f_{X,SL} \tag{4}$$

During the cornering process, apart from the tangential friction, a side friction must be available. The maximum value of the side friction coefficient ($f_{Y,MAX}$) may be considered to be identical to the sliding coefficient value in the longitudinal direction ($f_{X,SL}$). The distribution of friction in both directions as a vehicle undergoes a culvilinear motion is governed by the expression (*9*):

$$\left[\frac{f_Y}{f_{Y,MAX}}\right]^2 + \left[\frac{f_X}{f_{X,MAX}}\right]^2 \leq 1 \tag{5}$$

Equations 1 to 5 constitute the analytical tool for the operational design of a roadway alignment. By using those equations, the least value of a curve radius or the necessary superelevation for accomplishing safely the cornering motion at a given design speed [or 85th percentile operating speed (V_{85})] may be calculated. Inversely, the safe cornering speed can be determined for a curve with a given radius and superelevation rate. Those properties, which are already well known for the mass-point model and the braking mode at level alignments, are made available for the enriched bicycle vehicle model developed herein and the driving mode for a three-dimensional alignment.

QUANTITATIVE ANALYSIS

A quantitative analysis was carried out to determine the deviations that may be imposed on the design road parameters by the equations formulated above. In the following quantitative analysis, the numerical investigation is conducted by defining a representative car (representative values of various vehicle characteristics). Obviously, its characteristics may change from one country to another as well as over time. The characteristics of this car should be compatible with recent technological changes; however, the characteristics of the older cars that are still in use may mainly influence their prototype parameters. Furthermore, it should be stressed that such cars should have unfavorable characteristics to meet the safety criteria that are usually set up according to a considerably conservative threshold (*11*).

It has been proven that the characteristics that mainly influence vehicle performance in the cornering process are the vehicle drive, the vehicle mass (m), the aerodynamic drag coefficient (c), and the position of the vehicle's center of gravity along its longitudinal axis (l_R), whereas a minor influence is the height of the center of gravity above the pavement (h) (*1*). For the needs of the work described here, the following values are assigned to them: m = 1000 kgr, c = 0.4, $l_R/1$ = 0.4; and h/l = 0.25 (where l is the distance between the front and rear wheels; Front-wheel drive) (*12,13*). The present investigation is limited to curved segments with superelevation rate q = 0.07.

Comparison Between Design Policies with Different Safety Margins

The R_{MIN} values calculated for various alignments and design speeds are compared with the corresponding radii suggested by AASHTO-1990 and the German RAS-1984 geometric design policies for highways. The discrepancy in the numerical values is due to the different safety margins accepted by AASHTO-1990 and RAS-1984 in the values of available road-pavement friction. Specifically the RAS-1984 policies accept values considerably lower than accepted by ASSHTO-1990, which additionally decrease significantly with increasing speed [i.e., at (50 mph), $f_{X,SL}$ = 0.3 and 0.24 at 70 mph, $f_{X,SL}$ = 0.28 and 0.17 for AASHTO-1990 and RAS-1984, respectively]. It should be noted that the discrepancy between the maximum allowable tangential friction values found in the AASHTO-1990 policy and the RAS-1984 guidelines seems to originate from the pavement data inventory. Friction values depend on a variety of factors (type and condition of tires, type and condition of the pavement surface, weather conditions, vehicle and driver performance under driving or braking modes). The RAS-1984 friction values were determined on the basis of an extended data inventory (*10*) and correspond to the skid resistance values of 95 percent of new pavements in the Federal Republic of Germany. That means that only 5 percent of any new road surfaces may be invalid for the application of these values (*14*). On the contrary, the AASHTO-1990 policies admit that although all influencing factors should be incorporated in the determination of friction values, "*available* data are not fully detailed over the range for all those variables, and conclusions must be made in terms of the safest *reported* average values" (*1*).

Furthermore, an indication of an insufficient friction values inventory is that "American friction values . . . clearly contradict the worldwide research experience which shows that friction values should substantially decrease with increasing speed" (*15*).

Influence of Vehicle Speed

In the present analysis, the important aspect of vehicle speed is examined. Four major results are derived.

1. There is a relationship between the horizontal curve radius and the grade, whereas the latter is superimposed on the former at the same road segment (Figures 1 to 4). This finding opposes the classical approach in which these two elements are considered to be independent with respect to safety; thus, the horizontal configuration and the vertical profile are selected independently and without serious interaction.
2. The R_{MIN} increases with increasing grade; their relationship is described by a convex function (Figures 1 to 4). This finding appears to violate the intuition of the highway engineer, according to which the safety requirements become more critical as the vehicle moves downgrade. However this rule is valid when considering the braking mode, whereas for the vehicle motion in driving mode, the opposite is correct. In the latter case, as the vehicle moves upgrade, greater longitudinal forces act on it, demanding greater reserves of friction. Consequently, fewer reserves of friction remain to be used in the lateral direction (Tables 1 and 2).
3. The required R_{MIN} values increase dramatically with grade at higher vehicle speeds (Figures 3 and 4).
4. In a number of cases the values of R_{MIN} given by the existing guidelines are lower than those that are required. This means that they underdesign for these cases, because they do not consider the driving mode of the vehicle motion as being critical.

In general, drivers actually move at speeds higher than the design speed (*16,17*). The consistency of road alignment should not allow deviations of V_{85} more than 16 to 20 km/hr (10 to 12 mph) from the design speed. Even under favorable driving conditions, this criterion does not exclude the possibility of inadequate minimum horizontal curve radii. On the basis of a broad classification of highway alignment (very good, good, and fair), an investigation of the necessary R_{MIN} values for all three cases was carried out. This broad classification corresponds to an accepted deviation of V_{85} from design speeds of 0, 5, and 10 mph, respectively (*18*).

The result is shown in Figures 1 to 4 for design speeds 50 and 60 mph. The former reveals that AASHTO-1990 values for R_{MIN} are adequate for good geometric designs. For fair designs, however, values greater than the suggested values are needed for grades of more than 3 percent. Worse results are apparent in the case of RAS-1984 policies. The relatively conservative friction factors that are accepted can be exceeded even at a very good

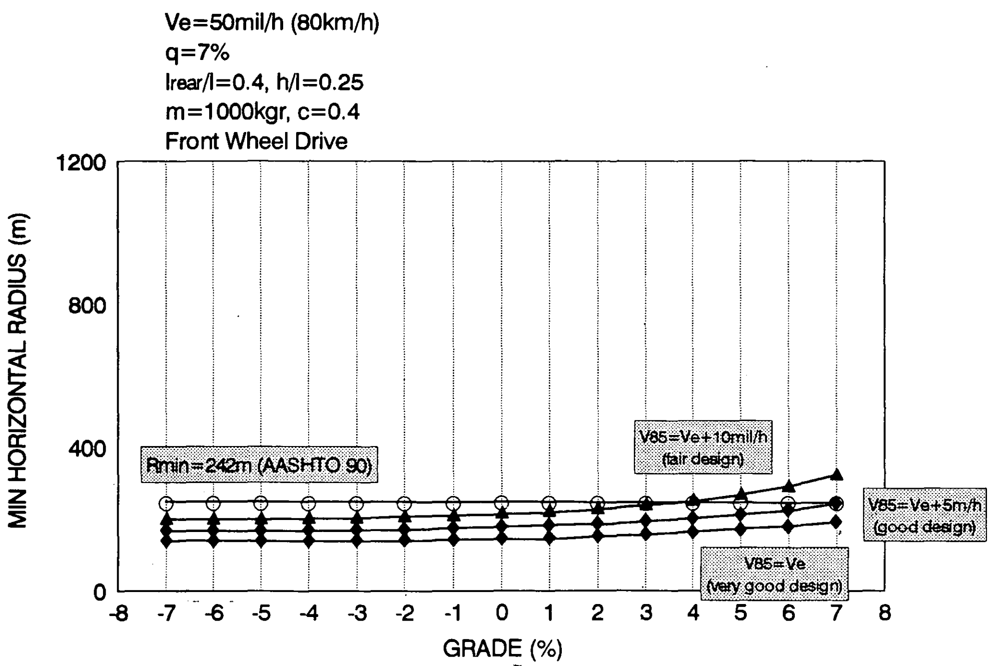

FIGURE 1 Influence of speed on R_{MIN} (V = 50 mph) values on the basis of AASHTO-1990.

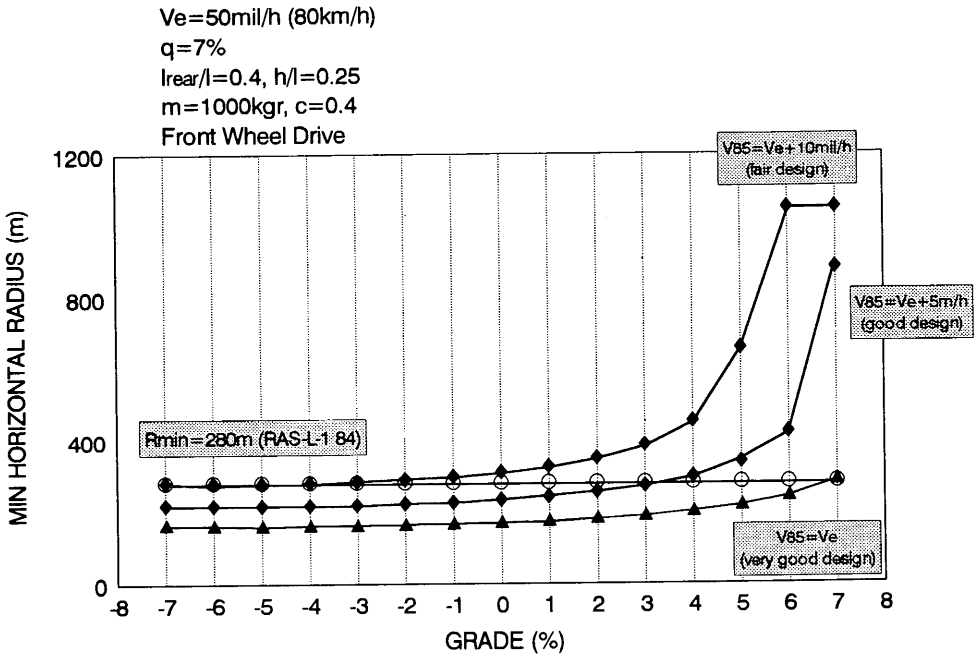

FIGURE 2 Influence of speed on R_{MIN} (V = 50 mph) values on the basis of RAS-1984.

geometric design when the operating speed equals the design speed (Figures 1 and 2). At a design speed of 60 mph, a fair geometric design worsens the situation dramatically. Figure 4 shows that at this speed the R_{MIN} values suggested by RAS-1984 lie well below the minimum values necessary in the driving mode at all grades.

The differences between design policies were derived, as mentioned above, because they assume different safety margins (*14*). Specifically, RAS-1984 assumes much worse pavement performance than that assumed by AASHTO-1990 as being representative, revealing in this way the more apparent underdesigning trend. In this sense, although it is stated that AASHTO-1990 leads

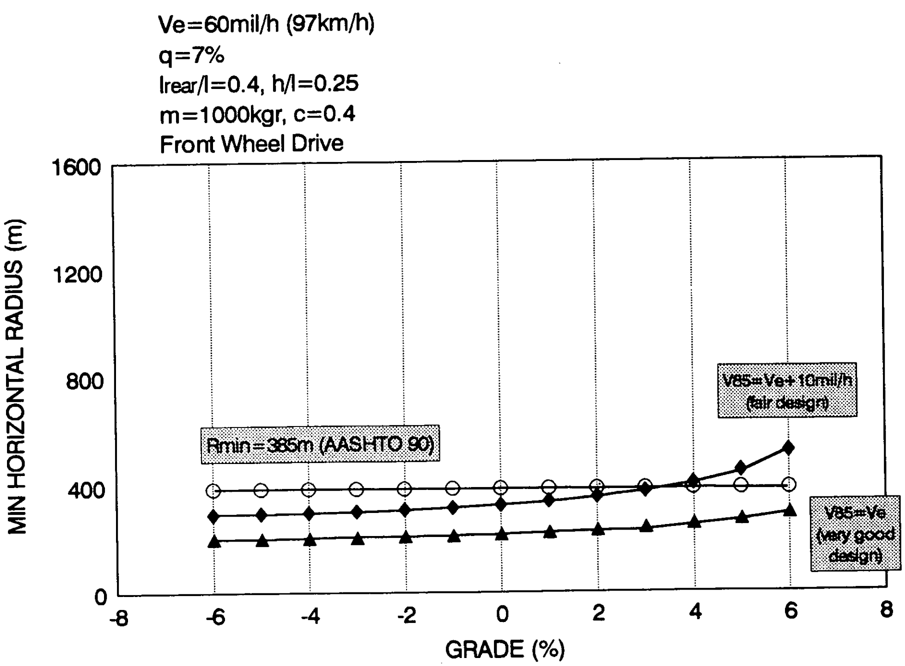

FIGURE 3 Influence of speed on R_{MIN} (V = 60 mph) values on the basis of AASHTO-1990.

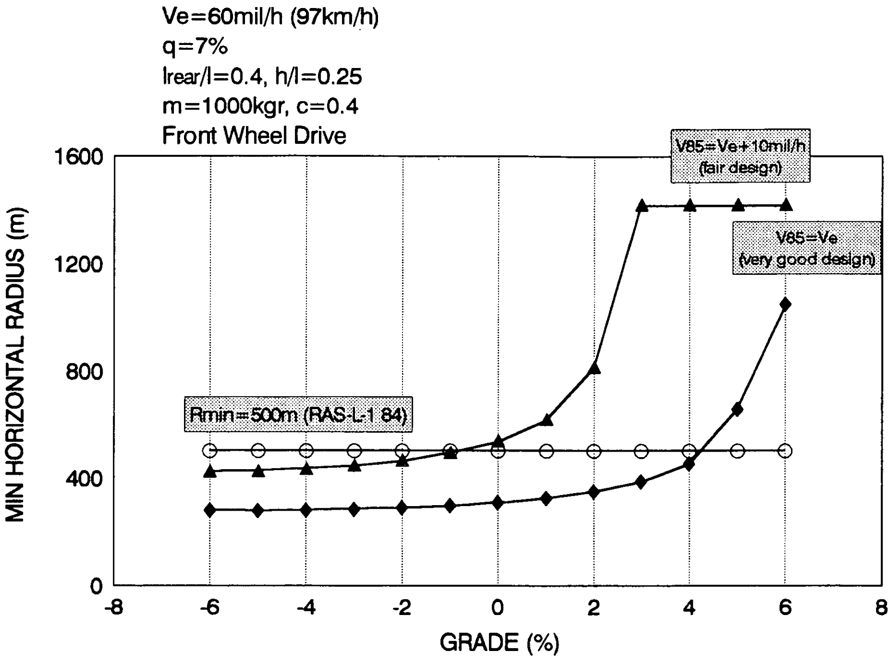

FIGURE 4 Influence of speed on R_{MIN} (V = 60 mph) values on the basis of RAS-1984.

TABLE 1 Demand Values of f_X for Various Speeds and Grades, and Remaining Friction Reserve To Be Used in Lateral Direction f_Y (Values Corresponding to AASHTO-1990 Policy); 1 mph = 1.61 km/hr)

Grade	V_{85} = 50mph		V_{85} = 60mph		V_{85} = 70mph	
	f_X	f_Y	f_X	f_Y	f_X	f_Y
0%	0.09	0.29	0.12	0.27	0.16	0.25
3%	0.17	0.27	0.20	0.25	0.24	0.21
6%	0.25	0.23	0.28	0.19	0.14	0.14

TABLE 2 Demand Values of f_X for Various Speeds and Grades and Remaining Friction Reserve To Be Used in Lateral Direction f_Y (Values Corresponding to RAS-1984 Policy)

Grade	V_{85} = 50mph		V_{85} = 60mph		V_{85} = 70mph	
	f_X	f_Y	f_X	f_Y	f_X	f_Y
0%	0.09	0.23	0.12	0.18	0.15	0.12
3%	0.17	0.20	0.19	0.13	0.23	0
6%	0.25	0.15	0.28	0	0.32	0

to underdesigns in fewer cases, this by no means should be interpreted as a safer policy standard than the RAS-1984 standard.

CONCLUSIONS AND RECOMMENDATIONS

The mass-point model and its inherent simplification in describing the cornering motion of a vehicle have deprived highway engineers of the ability to consider the phenomena governing the motion of a vehicle on a curve. In the preceding discussion, it was shown that neglect of the driving mode and the three-dimensional configuration of the roadway can lead to erroneous decisions concerning the selection of the appropriate horizontal curve radius. Design policies must recognize this fact. More striking results may be obtained and, consequently, a wider number of problematic situations may be identified if more conservative friction values are used. This is not a theoretical exercise, because recent as well as earlier researchers have pointed out (*19,20*) that in the driving mode the peak friction values cannot actually be used, let alone be used to determine safety criteria.

Furthermore, a new dynamic approach to the road design process must be introduced. The term *dynamic* is used in two ways. The first approach concerns the road design process itself, because the minimum horizontal curve radius does not remain constant along a roadway but changes with operating speed and grade of alignment. This can be termed the *internal dynamics* of the geometric design process. The second approach, *external dynamics*, refers to vehicle characteristics and pavement condition. Those characteristics generally vary from one country to another, thus implying different safety needs at curve sites. The deviating results obtained by using the AASHTO-1990 and the RAS-1984 values in the present analysis are a typical example of the different parameters of external dynamics in the road design process.

There is no doubt that the present analysis is deterministic. The intention of this paper, however, is not to give ready-to-use results but to point out critical safety situations that arise from the driving mode and the three-dimensional configuration of the road, which are not considered today. Further work is needed to integrate the stochastic dimension of the problem and satisfy the dynamic property of the road design process. A recent attempt applied to the mass-point model variables has been found in the literature (*21*); however, it may not be easy to repeat that work with the variables of the enriched model proposed in this paper. It may be difficult or even impossible to determine a probability distribution function of a variable such as R_{MIN}, depending on a number of stochastic independent variables like vehicle characteristics, speed, and pavement quality.

In addition, the following issues need to be investigated:

- The cornering performance of each individual vehicle type in the passenger car fleet must be examined and the results compared with current design policies.
- Because highways are built to serve the entire car fleet, an extension of the above investigation must be conducted to include trucks as well.
- On the basis of the two types of investigations needed as mentioned above, a comparison of the two vehicle modes, driving and braking, must be performed to determine which will govern the calculation of the critical value of a specific parameter for each combination of horizontal and vertical geometric elements.
- In the case of two-lane rural roads the operating speed has been proven to be a crucial design parameter beyond the design speed (*18,22*). Therefore, the analysis of the critical values of the design parameters must include the operating speed.

All these efforts are prerequisites before definite decisions for the minimum horizontal curve radius and other geometric features of the alignment can be made.

Finally, it should be pointed out that the analysis presented herein can be directly implemented in tort liability cases to determine the influence of vehicular parameters on the driving performance in a specific highway segment. This may prove to be of decisive importance under several circumstances.

APPENDIX A REPRESENTATION OF THREE-DIMENSIONAL ROAD SURFACE

The center of gravity of a vehicle (passenger car) is assumed to move on a space curve defined by its position vector **r**. To this curve (center of road line) is assigned a triplet of unit vectors (**t**, **n**, **b**), mutually orthogonal, composing the moving trihedron of the curve (*23*).

All forces and moments applied to a vehicle responsible for its movement in a trajectory, coinciding with the curve defined by the position vector **r**, may be expressed as a function of another triplet of unit vectors (t, ϵ, ζ) of the three-dimensional surface, with the curve given by **r** as its generator. Such surfaces are well known from differential geometry as *ruled surfaces*. The relationship that holds between the triplet (t, ϵ, ζ) and the conventional parameters that define a three-dimensional road surface, that is, cross-slope q and grade s, is in matrix form

$$(t, \epsilon, \zeta) = D(\hat{t}, \hat{\epsilon}, \hat{\zeta})^T \qquad \text{(A-1)}$$

where D is the transformation matrix:

$$D = \begin{vmatrix} 1 + \alpha qs & -\alpha & \alpha q - s \\ \alpha - qs & 1 & -\alpha s - q \\ s & q & 1 \end{vmatrix} \qquad \text{(A-2)}$$

and the unit vector triplet $(\hat{t}, \hat{\epsilon}, \hat{\zeta})$ represents the corresponding triplet to a plane surface (with no grade or cross-slope).

APPENDIX B VEHICLE DYNAMICS ON A THREE-DIMENSIONAL ROAD SURFACE

The motion of a passenger car on a road can be divided into three translatory movements, namely, longitudinal, lateral, and vertical, as well as three rotational movements, yaw, roll, and pitch. All of these individual movements occur along and around the vector triplet (t, ϵ, ζ). However, not all of them are important in terms of road design. Only the movements along and around the tangential vector t, that is, longitudinal movement and lateral movement, are critical for the formulation of road design criteria.

In considering the moving vehicle as the reference system, the forces imposed on it can be determined. These are illustrated in Figure B-1 (*24*).

1. The gross vehicle weight $\boldsymbol{G}$

$$G = mg\boldsymbol{\zeta} \tag{B-1}$$

2. The wheel-pavement contact forces analyzed to three components: the driving forces $\boldsymbol{X}$, the lateral forces $\boldsymbol{Y}$, and the vertical forces $\boldsymbol{Z}$, which for small slip angles are

$$\boldsymbol{X} = (X_F + X_R)\,\boldsymbol{t} \tag{B-2}$$

$$\boldsymbol{Y} = -\,(Y_F + Y_R)\,\boldsymbol{\epsilon} \tag{B-3}$$

$$\boldsymbol{Z} = -\,(Z_F + Z_R)\,\boldsymbol{\zeta} \tag{B-4}$$

3. The air resistance force $\boldsymbol{A}$

$$\boldsymbol{A} = -\,(A_X\boldsymbol{t} + A_Z\boldsymbol{\zeta}) \tag{B-5}$$

Under the influence of the above forces as well as of different moments (rolling resistance moments, bearing moments, etc.), the vehicle moves on the road surface, whereby the conservation laws of linear and angular momentum apply.

$$\sum_{\text{Forces}} = m\mathbf{r}'' = m\frac{dv}{dt}\boldsymbol{t} + m\frac{V^2}{R}\boldsymbol{n} + m\frac{V^2}{H}\boldsymbol{b} \tag{B-6}$$

and

$$\sum_{\text{Moments}} = D' \tag{B-7}$$

Introduction of Equations B-1 to B-5 into Equations B-6 and B-7 and successive multiplication by the vectors $\boldsymbol{t}$, ϵ, and ζ result in an explicit expression of the acting forces on the moving vehicle. Not all of them, in fact, are of particular interest to the road designer, as mentioned above. Taking into consideration a nonaccelerated movement of the vehicle on a helical surface (i.e., a three-dimensional surface: horizontal curve and constant grade) in which all three geometric parameters remain constant, namely, grade s, superelevation q, and radius R

$$V = \text{constant} \tag{B-8}$$

$$s \simeq \tan(s) \simeq \sin(s) = \text{constant} \tag{B-9}$$

$$q \simeq \tan(q) \simeq \sin(q) = \text{constant} \tag{B-10}$$

$$R = \text{constant} \tag{B-11}$$

the expressions giving the forces acting on a vehicle with front-wheel drive are obtained as follows:

$$X_F = Gs + A_X + \frac{m^2}{4\delta}\left[\frac{V^2}{R} - gq\right]^2 + f_R Z_R \tag{B-12}$$

$$X_R = -\,f_R Z_F \tag{B-13}$$

Corresponding expressions may be derived for a vehicle with a rear-wheel drive.

$$Y_F = m\left[\frac{V^2}{R} - gq\right]\frac{l_R}{l} \tag{B-14}$$

$$Y_R = m\left[\frac{V^2}{R} - gq\right]\frac{l_F}{l} \tag{B-15}$$

$$Z_F = G\left[\frac{l_R}{l} - s\frac{h}{l}\right] - A_{Z,F} + mq\frac{V^2}{R}\frac{l_R}{l} \tag{B-16}$$

$$Z_R = G\left[\frac{l_F}{l} + s\frac{h}{l}\right] - A_{Z,R} + mq\frac{V^2}{R}\frac{l_F}{l} \tag{B-17}$$

$$A_{Z,F} = A_Z\frac{l_R}{l} - A_X\frac{h}{l} \tag{B-18}$$

$$A_{Z,R} = A_Z\frac{l_F}{l} + A_X\frac{h}{l} \tag{B-19}$$

To obtain the above equations, the assumption was made that the axes of the front and rear wheels and the center of gravity are tracing nearly parallel curves. Furthermore, the coefficient of tire stiffness δ is considered equal for all tires, whereas the lateral force is linearly related to the slip angle α.

From the above set of expressions, the friction coefficients in the longitudinal and lateral directions are readily available. The friction coefficients are given as

$$f_{X,F} = \frac{X_F}{Z_F} \tag{B-20}$$

$$f_{Y,F} = \frac{Y_F}{Z_F} \tag{B-21}$$

Similar expressions may be used for the friction coefficients for the rear wheels.

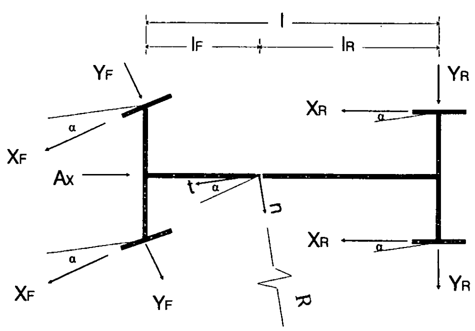

FIGURE B-1 Forces acting on a passenger car moving along a road segment.

NOMENCLATURE

A, A_X, A_Z, $A_{Z,F}$, $A_{Z,R}$ = Air resistance force and its components to respective axes and to respective wheels (F = front; R = rear)

$\boldsymbol{b}$ = Unit binormal vector

c = Aerodynamic drag coefficient
f_R = Rolling coefficient of friction
$f_{X,F}, f_{Y,F}, f_{X,R}, f_{Y,R}$ = Coefficients of friction to respective directions and wheels
$f_{X,\mathrm{MAX}}, f_{Y,\mathrm{MAX}}$ = Maximum coefficient of friction to respective directions
$f_{X,\mathrm{SL}}$ = Sliding coefficient of friction
H = Radius of vertical curve
h = Distance of center of gravity from the pavement
G = Vehicle weight
g = Gravity acceleration rate
l = Distance between front and rear wheels
l_F, l_R = Distance between the center of gravity and the respective wheels
m = Vehicle mass
$\mathbf{n}$ = Principal normal unit vector
n_F, n_R = Factors
q = Superelevation rate
R = Radius of horizontal curve
s = Grade
$\mathbf{t}$ = Unit tangent vector
V = Vehicle speed
V_{85} = 85th percentile of speed
X_F, X_R = Driving forces acting on respective wheels
Y_F, Y_R = Lateral forces acting on respective wheels
Z_F, Z_R = Vertical forces acting on respective wheels
α = Slip angle
δ = Tire stiffness coefficient
ϵ = Generating vector
ζ = Normal vector

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New and Improved Unsymmetrical Vertical Curve for Highways

SAID M. EASA

A new unsymmetrical vertical curve for highways that provides important desirable features is developed. The curve has unequal horizontal projections of the tangents, but its component parabolic arcs are equal. The new curve minimizes the difference between the rates of change of grades of the two arcs and consequently provides a smoother ride and is more aesthetically pleasing. The curve also improves the sight distance, reduces the length requirements, increases rider comfort, and increases the vertical clearance compared with the traditional unsymmetrical vertical curve. These desirable features should make the new unsymmetrical vertical curve an important element in vertical alignment design.

The traditional unsymmetrical vertical curve consists of two unequal parabolic arcs that meet at the tangents' intersection. The geometric characteristics of this curve have been presented in highway and surveying engineering texts (*1–3*). AASHTO points out that on certain occasions, because of critical clearance or other controls, unsymmetrical vertical curves may be required (*4,5*). However, because the need for these curves is infrequent, no information on them has been included by AASHTO.

Approximate relationships between the length of an unsymmetrical crest vertical curve and sight distance have been developed (*6*). These relationships assume that, for minimum sight distance, the line of sight is tangent to the point of common curvature and, consequently, may greatly underestimate the curve length requirements. Exact length requirements of crest and sag vertical curves that satisfy sight distance needs have been developed (*7,8*).

The traditional unsymmetrical vertical curve has some limitations caused by fixing the point of common curvature at the tangents' intersection. First, the difference between the rates of change of grades of the two arcs is generally large. As a result, the minimum available sight distance, which is controlled by the sharper arc, is short and the required curve is long. Second, the curve is not smooth and is less aesthetically pleasing. Third, for fixed ends of the unsymmetrical vertical curve, the required vertical clearance may not be satisfied and the curve is not suitable when it must pass through a fixed intermediate point.

In this paper a new unsymmetrical vertical curve that minimizes the difference between the rates of change of grades of the two arcs is developed. The improvements in sight distance, curve length requirements, rider comfort, and vertical clearance achieved by this curve are examined. Before presenting the new unsymmetrical vertical curve, the traditional unsymmetrical vertical curve is described.

TRADITIONAL UNSYMMETRICAL VERTICAL CURVE

A traditional unsymmetrical crest vertical curve (hereafter called the *traditional curve*) is shown in Figure 1. The curve consists of two parabolic arcs that have a common tangent at the point of common curvature (PCC). The PCC lies at the intersection of the two tangents, known as the point of vertical intersection (PVI). The beginning and end points of the vertical curve (BVC and EVC) have tangents with grades g_1 and g_2 (in percent), respectively. The algebraic difference in grade, A, equals $|g_2 - g_1|$. For crest vertical curves $(g_2 - g_1)$ is negative, and for sag vertical curves it is positive. The absolute value of $(g_2 - g_1)$ is used so that A is positive for both crest and sag vertical curves. The rates of change of grades of the two arcs are given by Hickerson (*1*)

$$r_1 = \frac{AL_2}{100\,LL_1} \tag{1}$$

$$r_2 = \frac{AL_1}{100\,LL_2} \tag{2}$$

where

r_1 = rate of change of grades of the first arc,
r_2 = rate of change of grades of the second arc,
A = algebraic difference in grade (in percent),
L_1 = length of the first arc,
L_2 = length of the second arc, and
L = total length of the curve $(L_1 + L_2)$.

Note that r_1 and r_2 will be positive for both crest and sag vertical curves. When $L_1 = L_2 = L/2$, Equations 1 and 2 give $r_1 = r_2 = A/100\,L$, which is the rate of change of grades of a symmetrical vertical curve, r. For an unsymmetrical vertical curve in which $L_1 < L_2$, for example, $r_1 > r$ and $r_2 < r$. This indicates that the first arc has a larger curvature (is sharper) than the symmetrical vertical curve, whereas the second arc has a smaller curvature (is flatter) than the symmetrical vertical curve.

A convenient parameter for describing the unsymmetrical vertical curve is R, which is defined as the ratio of the length of the shorter tangent (or shorter arc in the case of a traditional curve) to the total curve length,

$$R = \frac{L_1}{L} \tag{3}$$

Expressing Equations 1 and 2 in terms of R gives

$$r_1 = \frac{A(1 - R)}{100\,LR} \tag{4}$$

$$r_2 = \frac{AR}{100\,L(1 - R)} \tag{5}$$

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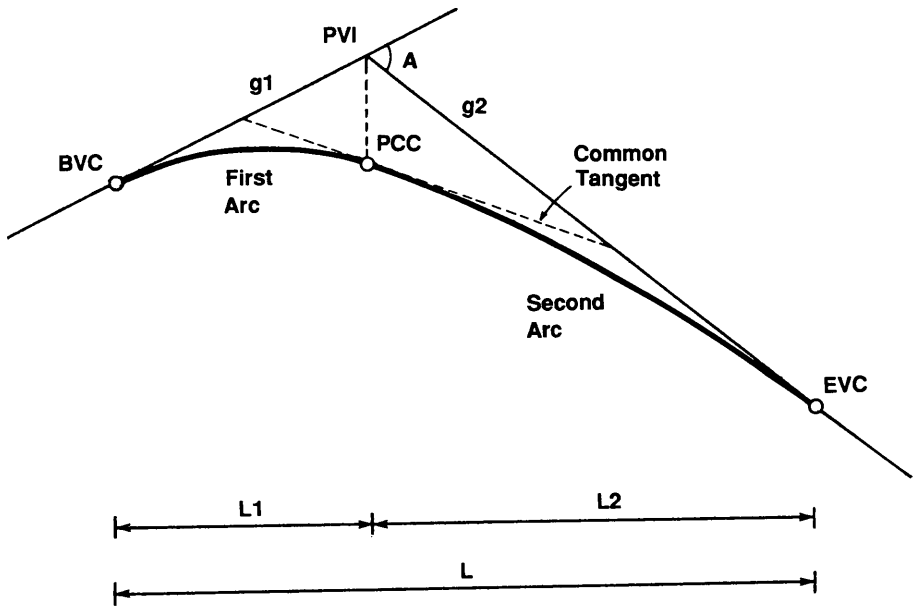

FIGURE 1 Traditional unsymmetrical crest vertical curve ($L_1 < L_2$).

For $R = 0.5$, Equations 4 and 5 give $r_1 = r_2 = A/100\ L$, and the traditional curve reduces to a symmetrical vertical curve.

NEW UNSYMMETRICAL VERTICAL CURVE

The new unsymmetrical vertical curve is derived from a general unsymmetrical vertical curve in which PCC is located at an arbitrary point. A description of the general and new unsymmetrical vertical curves follows.

General Unsymmetrical Vertical Curve

The geometry of a general unsymmetrical vertical curve is shown in Figure 2. PCC is located at a distance d_1 from BVC and a distance d_2 from EVC. Suppose that the horizontal projection of the tangent L_1 is less than L_2. The derivation of the rates of change of grades of the two arcs follows.

From Figure 2, the distances *ab*, *bc*, and *ac* are given by

$$ab = \frac{A}{100}(d_1 - L_1) \tag{6}$$

$$bc = \frac{r_2 d_2^2}{2} \tag{7}$$

$$ac = \frac{r_1 d_1^2}{2} \tag{8}$$

Since $ab = ac - bc$, then

$$\frac{A}{100}(d_1 - L_1) = \frac{r_1 d_1^2}{2} - \frac{r_2 d_2^2}{2} \tag{9}$$

Also, from Figure 2,

$$A_1 + A_2 = A \tag{10}$$

$$r_1 d_1 + r_2 d_2 = \frac{A}{100} \tag{11}$$

Solving Equations 9 and 11 for r_1 and r_2, and noting that $d_2 = L - d_1$, then

$$r_1 = \frac{A(L + d_1 - 2L_1)}{100\ Ld_1}, \quad L_1 < L_2 \tag{12}$$

$$r_2 = \frac{A(-d_1 + 2L_1)}{100\ L(L - d_1)}, \quad L_1 < L_2 \tag{13}$$

Equations 12 and 13 are applicable only if $L_1 < L_2$. If $L_2 < L_1$, r_1 and r_2 are given by

$$r_1 = \frac{A(L - 3d_1 + 2L_1)}{100\ Ld_1}, \quad L_2 < L_1 \tag{14}$$

$$r_2 = \frac{A(3d_1 - 2L_1)}{100\ L(L - d_1)}, \quad L_2 < L_1 \tag{15}$$

Note that when PCC lies at PVI ($d_1 = L_1$), the preceding equations for $L_1 < L_2$ and $L_2 < L_1$ reduce to Equations 1 and 2 of the traditional curve.

Derivation of New Curve

The variation of r_1 and r_2 of Equations 12 and 13 with d_1 is shown in Figure 3, which corresponds to a general unsymmetrical vertical curve with $g_1 = +2$ percent, $g_2 = -3$ percent, $L_1 = 250$ m, and $L = 800$ m. When $d_1 = 0$, $r_1 = \infty$ and r_2 has a finite value. As d_1

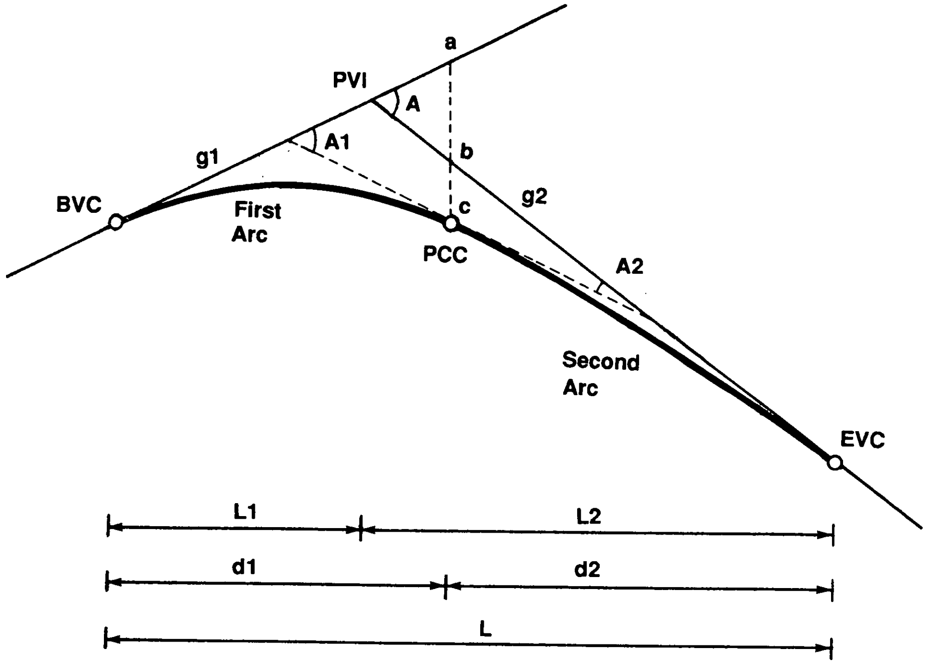

FIGURE 2 General unsymmetrical crest vertical curve ($L_1 < L_2$).

increases, both r_1 and r_2 decrease but r_1 is a convex function of d_1 and r_2 is a concave function of d_1. For $d_1 = 2L_1$, Equations 12 and 13 give $r_1 = A/200L_1$ and $r_2 = 0$, respectively. Therefore, values of d_1 equal to or greater than $2L_1$ are infeasible. The objective is to find d_1 that corresponds to the minimum difference between r_1 and r_2. Let the difference be denoted by F,

$$F = r_1 - r_2 \tag{16}$$

Substituting for r_1 and r_2 from Equations 12 and 13, Equation 16 can be expressed in terms of d_1 as

$$F = \frac{A(L - 2L_1)}{100\, d_1(L - d_1)} \tag{17}$$

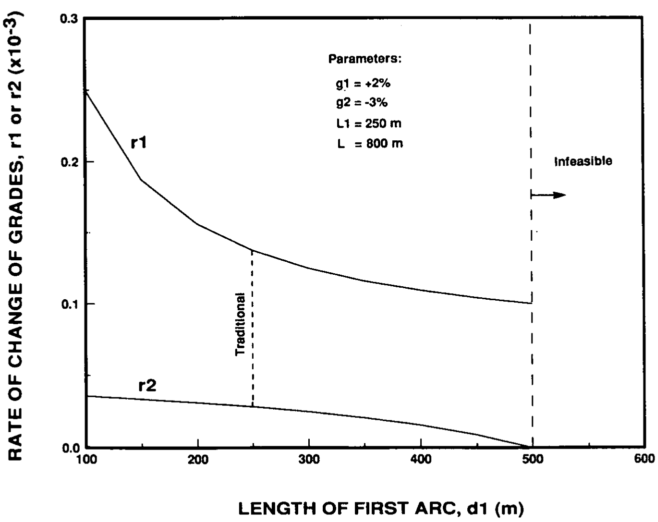

FIGURE 3 Variation of rates of change of grades with length of first arc.

The minimum value of F occurs when the first derivative of F with respect to d_1 equals zero. Differentiating both sides of Equation 17 and equating dF/dd_1 to zero yields

$$A(L - 2L_1)(L - 2d_1^*) = 0 \tag{18}$$

where d_1^* is the length of the first arc corresponding to the minimum value of F. On the basis of Equation 18, then

$$d_1^* = \frac{L}{2} \tag{19}$$

That is, the minimum value of F occurs when the two arcs of the unsymmetrical vertical curve are equal. This curve is referred to throughout as the *equal-arc unsymmetrical* (EAU) curve. The condition of Equation 19 corresponds to a minimum difference between r_1 and r_2 (not a maximum) because the second derivative of F can be shown to be always positive. Figure 4 shows the variation of F with the length of the first arc, d_1, and the minimum point, which occurs at $L/2$.

Substituting for $d_1 = L/2$ into Equations 12 and 13, r_1 and r_2 of the EAU curve are obtained as

$$r_1 = \frac{A(3L - 4L_1)}{100\, L^2}, \qquad L_1 < L_2 \tag{20}$$

$$r_2 = \frac{A(-L + 4L_1)}{100\, L^2}, \qquad L_1 < L_2 \tag{21}$$

Expressing Equations 20 and 21 in terms of R of Equation 3 gives

$$r_1 = \frac{A(3 - 4R)}{100\, L}, \qquad L_1 < L_2 \tag{22}$$

$$r_2 = \frac{A(-1 + 4R)}{100\, L}, \qquad L_1 < L_2 \tag{23}$$

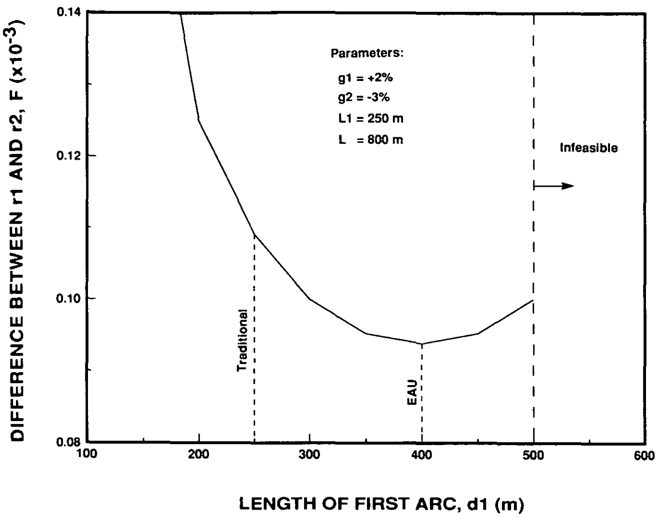

FIGURE 4 Variation of F with length of first arc.

For $L_2 < L_1$, Equations 22 and 23 are applicable, where $R = L_2/L$. Note that for $R = 0.5$, Equations 22 and 23 give $r_1 = r_2 = A/100\,L$ and the EAU curve becomes a symmetrical vertical curve. The traditional and EAU curves are drawn in Figure 5 for $L_1 = 250$ m, $L = 800$ m, $g_1 = +2$ percent, and $g_2 = -3$ percent. A symmetrical vertical curve with the same length as the unsymmetrical vertical curve is also shown.

Clearly, the EAU curve is smoother because the difference between r_1 and r_2 is minimal. The EAU curve is also more aesthetically pleasing because it not only reduces the difference between r_1 and r_2 but also makes the transitions at BVC and EVC less abrupt. This is true because the larger rates of vertical curvature of the two parabolic arcs of the EAU curve are more harmonious with the tangents whose rates of vertical curvature are infinity. The effect of the EAU curve on aesthetic appearance somewhat resembles the effect of a transition (spiral) curve on horizontal alignment.

PRACTICAL CONSIDERATIONS

Besides being smoother and more aesthetically pleasing, the EAU curve improves sight distance, requires a shorter length to satisfy a specific sight distance, increases rider comfort, and increases vertical clearance above that of a traditional crest vertical curve or below that of a traditional sag vertical curve. These benefits are quantified next.

Improving Sight Distance

The EAU curve improves the minimum sight distance compared with that of the traditional curve because both r_1 and r_2 of the EAU curve are smaller. Although exact models for computing the sight distance on the EAU curve are not available, the magnitude of the improvement can be approximately quantified. By using an idea by Guell (*9*), the minimum available sight distance and length requirements can be found on the basis of the sharper arc of the unsymmetrical vertical curve.

For the traditional curve, the minimum required length is

$$L = \frac{KA(1 - R)}{R} \quad \text{(traditional curve)} \tag{24}$$

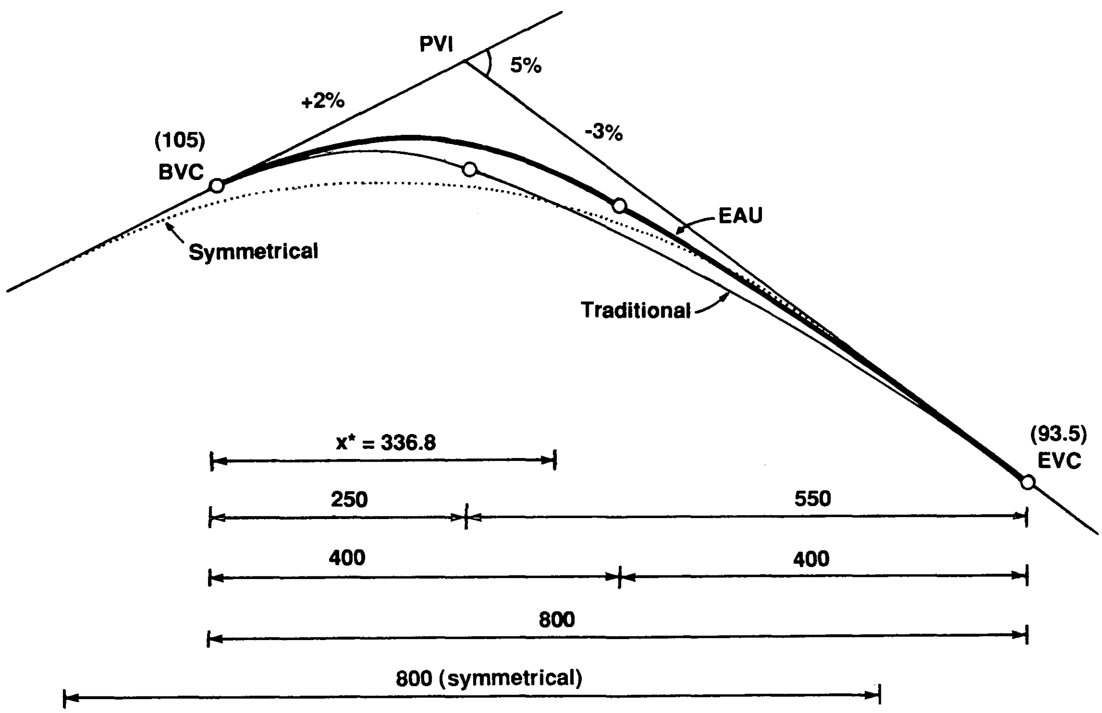

FIGURE 5 Comparison of traditional and EAU crest vertical curves (units in meters).

where K is the required rate of vertical curvature, which is the horizontal distance in meters (or feet) required to effect a 1 percent change in the grade, as given in AASHTO tables (5). For a crest curve, K is given by

$$K = \frac{S_m^2}{100\,(\sqrt{2h_1} + \sqrt{2h_2})^2} \tag{25}$$

where S_m = required minimum sight distance and h_1 and h_2 = the driver's eye and object heights, respectively. From Equations 24 and 25

$$S_m = (\sqrt{2h_1} + \sqrt{2h_2})\left[\frac{100\,LR}{A(1 - R)}\right]^{1/2} \quad \text{(traditional curve)} \tag{26}$$

For the EAU curve, the sight distance is controlled by the first (sharper) arc, which is true only when $L_1 < L_2$. Since $r_1 = 1/100K$, where r_1 is given by Equation (22), then

$$K = \frac{L}{A(3 - 4R)} \tag{27}$$

from which L and S_m are

$$L = KA(3 - 4R) \quad \text{(EAU curve)} \tag{28}$$

$$S_m = (\sqrt{2h_1} + \sqrt{2h_2})\left[\frac{100\,L}{A(3 - 4R)}\right]^{1/2} \quad \text{(EAU curve)} \tag{29}$$

The percentage increase in S_m achieved by the EAU curve, on the basis of Equations 26 and 29, can be obtained as

$$\text{Percent increase in } S_m = 100\left\{1 - \left[\frac{R(3 - 4R)}{(1 - R)}\right]^{1/2}\right\} \tag{30}$$

Table 1 shows the percentage increase in S_m for various values of R. The increase in S_m is greater for smaller R and reaches 26 percent for R = 0.2. For R = 0.5 both the traditional and EAU curves become a symmetrical vertical curve, and therefore no increase exists.

Reducing Curve Length Requirements

The required length of the EAU curve that satisfies a given sight distance (or K) is less than that of the traditional curve. The reduction in curve length, on the basis of Equations 24 and 28, can be obtained as

$$\text{Percent reduction in curve length} = \frac{100(1 - 2R)^2}{(1 - R)} \tag{31}$$

Table 2 shows the percentage reduction in L for various values of R. The reduction in curve length achieved by the EAU curve is significant and reaches 45 percent for R = 0.2.

For example, find the required length of an EAU crest vertical curve with R = 0.3 to satisfy stopping sight distance on a highway with g_1 = +2 percent, g_2 = −4 percent, and an 80-km/hr (50-mph) design speed. For this crest vertical curve, $A = |-4-2| = 6$ percent. From AASHTO (5), K = 36.70 m (120.39 ft) and the minimum required length, on the basis of Equation 28, is 397 m. For comparison, the minimum required length of the traditional unsymmetrical vertical curve is 514 m (the EAU curve length is 23 percent less).

Increasing Rider Comfort

The comfort effect caused by a change in vertical direction is greater on sag than on crest vertical curves because the centrifugal vertical force and the gravitational force are combining rather than opposing forces (5). The EAU curve reduces the centrifugal vertical acceleration on both sag and crest vertical curves and therefore increases comfort, especially on sag vertical curves. The centrifugal vertical acceleration equals the square of the design speed divided by the rate of vertical curvature,

$$C = \frac{V^2}{1{,}300\,K} \tag{32}$$

where

C = centrifugal vertical acceleration (m/sec^2),
V = design speed (km/hr), and
K = rate of vertical curvature (m/percent change of the grade).

By substituting for K of the traditional and EAU curves from Equations 24 and 27, respectively, into Equation 32, the corresponding centrifugal vertical accelerations on the first (sharper) arcs are

$$C = \frac{V^2A(1 - R)}{1{,}300\,LR} \quad \text{(traditional curve)} \tag{33}$$

$$C = \frac{V^2A(3 - 4R)}{1{,}300\,L} \quad \text{(EAU curve)} \tag{34}$$

TABLE 1 Increase in S_m Achieved by EAU Curve

R	Increase in Sm(%)
0.20	26
0.25	18
0.30	12
0.35	7
0.40	3
0.45	1
0.50	0

TABLE 2 Reduction in Curve Length Achieved by EAU Curve

R	Reduction in L(%)
0.20	45
0.25	33
0.30	23
0.35	14
0.40	7
0.45	2
0.50	0

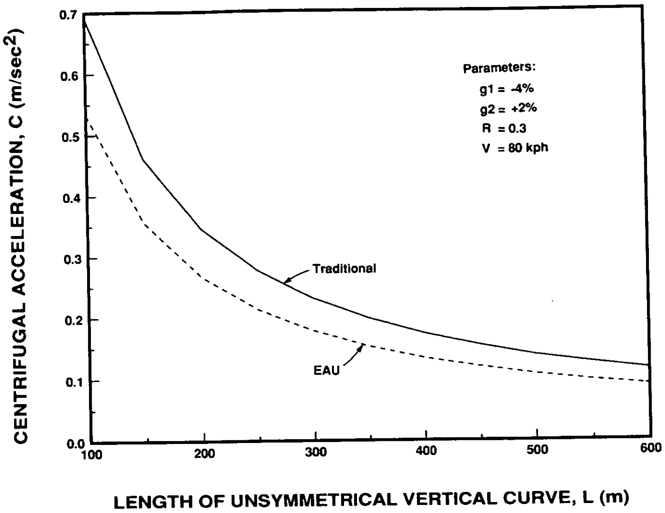

FIGURE 6 Comparison of centrifugal vertical accelerations of traditional and EAU sag vertical curves.

The centrifugal vertical acceleration of the EAU curve is smaller than that of the traditional curve. The percent reduction in C achieved by the EAU curve is given by the right side of Equation 31. Figure 6 shows the variations of C with L for the traditional and EAU curves, for V = 80 km/hr (50 mph), A = 6 percent, and R = 0.3. For this value of R, the centrifugal vertical acceleration on the EAU curve is 23 percent less than that on the traditional curve for any given L.

AASHTO points out that riding is comfortable on sag vertical curves when the centrifugal vertical acceleration does not exceed 0.3 m/sec^2 (1 ft/sec^2). The length of sag vertical curve that satisfies this comfort factor is much less than the headlight sight distance requirement. Therefore, the headlight criterion is used by AASHTO for the design of sag vertical curves (*5*). In Canada, the Roads and Transportation Association of Canada (RTAC) recommends the use of the comfort criterion for computing the length of sag vertical curve when good street lighting normally associated with urban conditions prevails (*10*). Under these conditions, sharper curves can be introduced and comfort is the criterion that limits values. Suostituting for C = 0.3 m/sec^2 in Equations 33 and 34, the minimum required lengths of the traditional and EAU sag vertical curves on the basis of the comfort criterion are

$$L = \frac{V^2A(1 - R)}{390\,R} \quad \text{(traditional sag curve)} \qquad (35)$$

$$L = \frac{V^2A(3 - 4R)}{390} \quad \text{(EAU sag curve)} \qquad (36)$$

For R = 0.5, Equations 35 and 36 reduce to $L = V^2A/390$, which is the minimum required length of symmetrical sag vertical curves on the basis of the comfort criterion (*10*). The percent reduction in sag vertical curve length achieved by the EAU curve is given by the right side of Equation 31.

Increasing Vertical Clearance

The EAU curve provides a larger vertical clearance, as shown in Figure 5. The maximum difference in vertical clearance occurs between the first arc of the EAU curve and the second arc of the traditional curve. The derivation of the maximum difference and its location for a crest vertical curve follows.

The elevations of the first arc of the EAU curve and the second arc of the traditional curve at a distance x from BVC are (Figure 7)

$$\text{Elevation (EAU)} = y_{\text{BVC}} + \frac{g_1x}{100} - \frac{r_1x^2}{2} \qquad (37)$$

$$\text{Elevation (traditional)} = y_{\text{EVC}} - \frac{g_2(L - x)}{100} - \frac{r_2(L - x)^2}{2} \qquad (38)$$

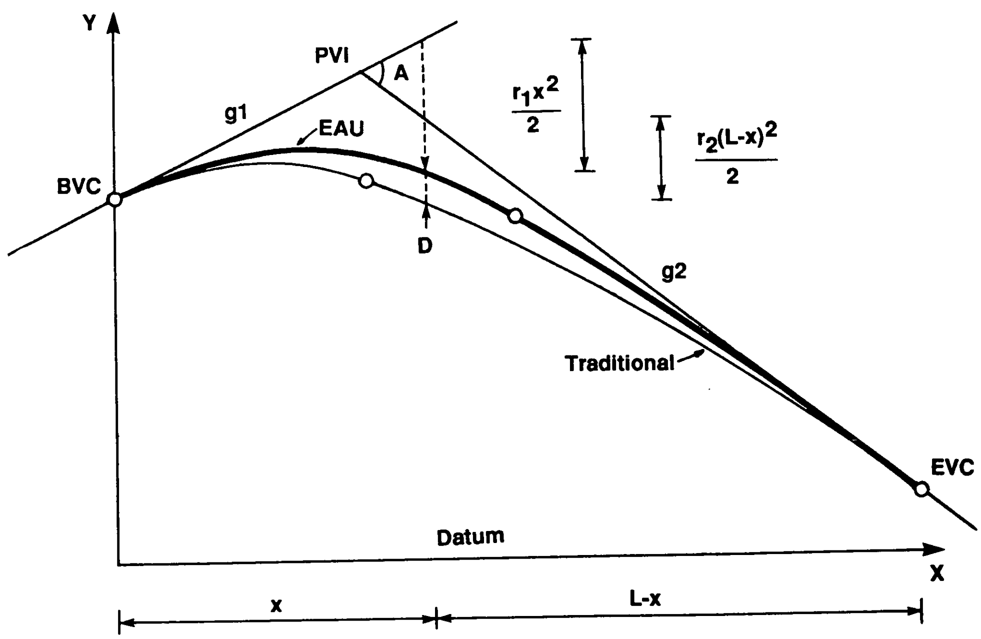

FIGURE 7 Geometry of increased vertical clearance of EAU crest curve.

Substituting for r_1 and r_2 from Equations 22 and 5, respectively, the difference between the elevations of the EAU and traditional curves, D, is

$$D = y_{BVC} - y_{EVC} + \frac{g_2 L}{100} - \frac{Ax}{100} - \frac{A(3 - 4R)x^2}{200\ L} + \frac{AR(L - x)^2}{200\ L(1 - R)} \quad (39)$$

Differentiating both sides of Equation 39 with respect to x, equating dD/dx to zero, and solving for x gives

$$x^* = \frac{L}{(3 - 2R)} \quad (40)$$

where x^* is the horizontal distance corresponding to the maximum difference D^*. Substituting for x^* into Equation 39 gives D^* as

$$D^* = y_{BVC} - y_{EVC} + \frac{g_2 L}{100} + \frac{AL(1 + 2R)}{200\ (3 - 2R)} \quad \text{(crest curve)} \quad (41)$$

where y_{BVC} and y_{EVC} = elevations of BVC and EVC, respectively. For sag vertical curves, Equation 40 is applicable and Equation 41 becomes

$$D^* = y_{BVC} - y_{EVC} + \frac{g_2 L}{100} - \frac{AL(1 + 2R)}{200\ (3 - 2R)} \quad \text{(sag curve)} \quad (42)$$

where D^* will be negative, indicating that the EAU curve lies below the traditional curve.

For example, find the maximum difference in vertical clearance between the EAU and traditional crest vertical curves of Figure 5, where $g_1 = +2$ percent, $g_2 = -3$ percent, $L_1 = 250$ m, $L = 800$ m, $y_{BVC} = 105$ m, and $y_{EVC} = 93.5$ m. For $A = |-3-2| = 5$ percent and $R = L_1/L = 0.3125$, the maximum difference in vertical clearance occurs at $x^* = 336.8$ m and its value is $D^* = 1.18$ m on the basis of Equations 40 and 41, respectively.

CONCLUDING REMARKS

In this paper a new unsymmetrical vertical curve in which the point of common curvature lies at the midpoint of the curve has been described. This EAU vertical curve has two important features: the rates of change of grades of the two arcs are less than those of the traditional unsymmetrical vertical curve and the difference between them is minimal. On the basis of the present study, the following concluding remarks are offered:

1. The EAU curve provides the following practical benefits: (a) improved sight distance, (b) reduced curve length requirements, (c) increased rider comfort, (d) increased vertical clearance, and (e) a more aesthetically pleasing curve.
2. The results show that the required length of the EAU curve that satisfies a specified sight distance is significantly shorter than that of the traditional curve. As a result, when a traditional crest curve lies totally in cut, the shorter EAU crest curve would reduce the cost of excavation. Likewise, when a traditional sag curve lies totally on fill, the shorter EAU sag curve would reduce the cost of fill and compaction. In other situations, the reduction in construction costs would depend on whether the EAU curve fits the terrain better than the traditional curve.
3. The EAU crest curve provides additional clearance over the traditional crest curve. Similarly, the EAU sag curve provides additional clearance below that of the traditional sag curve. This additional clearance would be useful when the vertical clearance restriction is below that of a traditional crest curve or above that of a traditional sag curve. If the vertical clearance restriction is in the opposite direction of the cases noted above, a symmetrical vertical curve should be considered.
4. The general unsymmetrical vertical curve, from which the EAU curve was derived, may also be useful on certain occasions. Because the location of PCC is a variable in this general curve, the curve offers flexibility in satisfying additional design constraints, such as the need for the curve to pass through a fixed point. A specific curve may also be selected to reduce earthwork or other construction costs. In this analysis, it is desirable to consider the curves that provide improvements over the traditional curve ($d_1 > L_1$) and not those curves that increase the difference between r_1 and r_2 ($d_1 < L_1$).
5. The sight distance and length requirements presented in this paper are approximate for $S_m > L_1$ (generally when A is small and when A is large and S_m is small). Exact sight distance models for the EAU crest and sag curves are currently being developed by the author.

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Three-Dimensional Analysis of Sight Distance on Interchange Connectors

EDDIE SANCHEZ

The design of interchange ramps and connectors, especially in large freeway-to-freeway interchanges, involves the use of stopping sight distance (SSD) criteria to determine horizontal and vertical geometries. Long connectors are usually required to avoid difficult horizontal and vertical obstructions. Therefore the use of minimum design standards for both horizontal and vertical geometries is quite common. The results of an investigation that evaluated SSD on interchange connectors by computerized three-dimensional (3-D) models are documented. Interchange connector models were developed by using combinations of minimal horizontal and vertical geometries with a longitudinal traffic barrier and a cross slope. A graphical procedure was used to measure SSD in a 3-D environment. The results revealed that the 3-D method of measuring SSD was not significantly different from the conventional two-dimensional method of measuring SSD. When all the 3-D models were rotated and viewed from different angles, the line of sight was always obstructed by the longitudinal barrier. Driver perspective views revealed that the cross slope affected the available SSD significantly. Therefore when a vertical curve is combined with a horizontal curve that requires a cross slope, the line of sight is not blocked by the roadway surface. This observation indicates an additional conservatism in the current crest vertical curve methodology. Designers should consider using computerized 3-D models in their normal design procedures. The use of models will allow designers to view different geometric configurations before deciding on the final combination. The models will allow designers to see the end result before the actual construction begins and thereby possibly eliminate costly field alterations.

The design of ramps and connectors is critical to the successful operation of a directional interchange. Direct connectors are usually long and geometrically complicated to avoid horizontal and vertical obstructions. Therefore, sight distance plays a critical role in the determination of a safe design. Typically designers develop horizontal and vertical geometries independently of one another on the basis of AASHTO sight distance requirements (*1*). When conservative values are used, the concern for the effect of their combination on sight distance is not generally an issue. With ever increasing traffic volumes and construction costs, a better understanding of the interactive effects of horizontal and vertical alignments on sight distance is needed to provide designers with the tools required to evaluate a variety of tight geometric combinations.

PROBLEM STATEMENT

Sight distance is the most basic and critical element of highway design. It controls all aspects of design from the establishment of alignments to the development of the cross-section elements of the roadway (*2*). AASHTO generally describes sight distance as the length of roadway ahead visible to the driver. Most designers use values greater than the minimum described by AASHTO. However, interchanges provide the designer with an interesting challenge of providing the maximum amount of sight distance while using minimum design values of horizontal and vertical geometries in combination.

According to AASHTO, horizontal and vertical alignments should complement each other to improve appearance and encourage uniform speed; however, poorly designed combinations can spoil the good points and aggravate the deficiencies of each (*1*). Designers have historically been trained to develop horizontal and vertical alignments independently. They must then depend on their ability to envision the roadway in perspective on the basis of the plan and profile views of the roadway (*3*). Therefore the task of producing complementary alignments should be assigned to designers with many years of experience. However, experience alone does not guarantee the most appropriate alignment combination.

The effect of inadequate stopping sight distance (SSD) on horizontal curves with minimum shoulder widths along longitudinal concrete traffic barriers is another area of concern for designers. According to a 1989 study by Leisch (*4*), SSD is not provided when curvature exceeds approximately 50 to 70 percent of the maximum curvature for a specified design speed on a roadway with 3.05-m (10-ft) shoulders. The paper by Leisch documented the results of a study that evaluated the effect of AASHTO Figure III-26A on horizontal sight distance in freeway and interchange reconstruction. Inadequate lateral offset of longitudinal concrete traffic barriers appears to be a common problem on interchange connectors. The problem is also compounded when severe crest vertical curves are also present.

AASHTO describes SSD as the minimum sight distance available on a roadway that would enable a below-average operator to stop a vehicle traveling at or near the design speed before reaching a stationary object in its path. AASHTO developed a model for determining the minimum amount of SSD based on the sum of the distance traveled by the vehicle during the perception-reaction time and the distance traveled during braking. The following equation is used to calculate SSD:

$$S = 1.47 * t_{pr} * V + \frac{V^2}{30(f + G)} \qquad (1)$$

where

S = SSD (ft),
t_{pr} = perception-reaction time (sec),
V = vehicle operating speed (mph),
f = coefficient of tire-pavement friction, and
G = roadway grade (decimal, + for upgrade, − for downgrade).

Texas Department of Transportation, P.O. Box 29928, San Antonio, Tex. 78284-3601.

AASHTO also describes sight distance as the distance along a roadway that an object of specified height is continuously visible to the driver. To measure this distance, three more parameters are required: (a) height of the driver's eye above the roadway surface, (b) the specified object height above the roadway surface, and (c) the height of the sight obstruction within the line of sight.

This methodology, which was formally introduced by AASHO in 1940, represented a significant change from the previous practice. The model introduced the concept of a small object [101.60-mm (4-in.)-high object] as the feature in the roadway rather than providing sight distance for the driver to see other vehicles in sufficient time to avoid them. Even though the model has remained the same, the parameters have shown a continuous change toward safer values. For example the driver eye height has been reduced to 1.07 m (3.5 ft) from the original 1.37 m (4.5 ft), and the pavement frictional values have been reduced approximately 70 percent (*5*). The assumed perception-reaction time is the only value that has remained constant from the original 1940 values.

DESIGN APPROACH

The design of highways has always been recognized as a three-dimensional (3-D) problem. Unfortunately, the current approach in determining horizontal alignments independently of vertical alignments requires designers to develop 3-D pictures in their minds simply by viewing two-dimensional (2-D) plans (plans, profiles, and cross sections). Although many experienced designers have no difficulty in visualizing an interchange from a 2-D drawing, problems arise when less experienced people have the design responsibility (*6*).

Computer-generated perspective plots have been used for a number of years to assist designers in visualizing details in the roadway. In 1968, Geissler (*3*) developed a computer program that developed perspective movies by feeding 3-D coordinates of terrain points into the computer and transforming them at desired intervals into perspective drawings that were finally photographed. In the same year, Park et al. (*7*) also developed a computer algorithm that plotted perspective views from the driver's eye by using design information as the data. This was the first attempt to develop 3-D perspectives from the driver's viewpoint. However, the process was very expensive because of the computer time and the amount of plotting required.

AASHTO's recommended method for developing good alignment coordination does not offer any quantitative criteria. Designers, experienced or not, must rely on their judgments in determining appropriate combinations of minimum horizontal and vertical alignment values. Although they are not recommended for interchanges, the use of near-minimum design values is common practice for connectors because of the limited availability of right-of-way and soaring construction costs. This paper attempts to provide insight into the effect of sight distance when using minimum design values of horizontal and vertical geometries in combination (*8*).

STUDY DESIGN

A typical one-lane bridge connector was developed and investigated for available sight distance. The focus of this paper was on the geometry of the horizontal and vertical alignment features in addition to the cross-section elements of the connector. The typical bridge connector was developed with a 3.66-m (12-ft) travel lane, a 1.22-m (4-ft) left shoulder, and a 2.44-m (8-ft) right shoulder. The Texas design manual (*9*) recommends a minimum 4.27-m (14-ft) travel lane for one-lane connectors with a minimum 1.22-m (4-ft) left shoulder. For sight distance evaluation, the critical issue was not the lane width but the distance from the centerline of the travel lane to the inside face of the barrier (*M*-lateral offset).

A test matrix, which included a range of values for the radius (*R*), the rate of change in vertical curvature (*K*) for the range of values of algebraic difference in grade (*A*), and the shoulder width (*M*-lateral offset), was developed to track any possible changes in the available sight distance. Table 1 lists the range of values for *R* based on *M* and AASHTO SSD values. The values of *M* ranged from 2.44 m (8 ft) to 5.49 m (18 ft) to cover as many possible combinations of lane and shoulder widths. The minimum 2.44-m (8-ft) value for *M* [0.61-m (2-ft) shoulder] was determined for those connectors constructed before the use of the criteria established in the Texas design manual. The 5.49-m (18-ft) value for *M* was chosen to be large enough to accommodate a 3.05-m (10-ft) shoulder adjacent to a 3.66-m (12-ft) travel lane. The listed radius (*R*) values were determined by using the following equation that relates the radius, the obstruction, the observer, and the object.

$$M = R\left[1 - \cos\left(\frac{90 * S}{\pi * R}\right)\right] = R\left[1 - \cos\left(\frac{28.65 * S}{R}\right)\right] \quad (2)$$

where

M = middle ordinate of curve (ft),
R = radius of curve (ft), and
S = stopping sight distance (ft).

TABLE 1 Values of Minimum Radius on the Basis of *M* and SSD

	40.23 km/h Design Speed (25 mph)	56.32 km/h Design Speed (35 mph)	72.42 km/h Design Speed (45 mph)
M (m - ft)	45.72 m SSD (150 ft)	68.58 m SSD (225 ft)	99.06 m SSD (325 ft)
2.44 - 8	106.68 - 350	240.79 - 790	502.62 - 1649
3.05 - 10	85.34 - 280	192.33 - 631	402.03 - 1319
4.27-14	60.66 - 199	137.16 - 450	286.82 - 941
5.49 - 18	51.82 - 170[a]	106.38 - 349	222.50 - 730

[a] Minimum Allowable Radius for e=0.08, M=4.96 m (16.27 ft)

The only exception to Table 1 was the M value of 5.49 m (18 ft) at a design speed of 40.23 km/hr (25 mph). According to Equation 2, this combination of M and SSD would equate to a radius of 46.68 m (153.152 ft). AASHTO requires the minimum radius for a design speed to be based on the maximum allowable side friction. Therefore according to AASHTO, the minimum allowable radius for a 40.23-km/hr (25-mph) design speed and a superelevation rate of 0.08 is 51.82 m (170.068 ft). By using Equation 2, this results in an M value of 4.96 m (16.27 ft). This relationship indicates that, for a design speed of 40.23 km/hr (25 mph), a superelevation rate of 0.08, and an M value of greater than 4.96 m, the minimum radius based on the side friction rather than the minimum radius based on SSD controls the design. A superelevation rate of 0.08 was used for all models because it is the recommended maximum allowed by the Texas design manual.

A total of 48 different 3-D computerized models were developed by using an interactive computer program on an Intergraph 225 MicroStation workstation and by using a software program called InRoads (*10*). The software program developed 3-D files on the basis of operator-developed alignment geometry and roadway surface templates to develop digital terrain models (DTMs) of the natural ground and the proposed connector surface. The DTM consisted of x, y, and z coordinates that were connected into 3-D triangular planes by using an algorithm known as Delauney's criterion. This criterion determined the smallest or most logical triangular surface on the basis of operator qualifiers (*10*).

The crest vertical curve lengths for all of the models were placed within the limits of the PC and the PT of the horizontal curve. The models were also developed with a vertical face longitudinal barrier on both sides of the roadway surface. This would simplify the location of conflict when determining the obstruction (roadway surface or barrier) hindering the line of sight. It was assumed that this combination would create the most complex geometric configuration.

The Intergraph 225 MicroStation workstation allowed the operator to develop perspective views of the 3-D models from any desired location. For each model a line was placed from the location representing the driver's eye [1.07 m (3.5 ft) above the roadway surface] and the object [152.4 mm (6 in. high)]. The models were then viewed from the top, the side, and then the driver's perspective looking at the object. From these views the operator was able to determine the obstruction impeding the line of sight.

The models were also used to calculate x, y, and z coordinates along the center of the travel lane. The distance between each coordinate was then calculated and summed by using a spreadsheet program. The summed distance thus represented the actual 3-D measurement of SSD along the roadway, which was then compared with the required 2-D SSD, which was measured on a flat horizontal plane.

RESULTS AND FINDINGS

Model Development

For the present study 12 independent horizontal alignments were developed to generate the 48 computer models. A typical alignment consisted of two tangent sections with a beginning point, a middle PI point, and an ending point. The same x and y coordinates were used to locate the beginning of the alignment, the PI, and the ending point for each alignment. The only difference in the alignments was the radius of the horizontal curve located at the PI, which was obtained from the required radius values in Table 1. A deflection angle of 90 degrees was also assumed for the horizontal alignments. The beginning station for the alignments was set at station 10+00; however, the ending station varied depending on the radius of the horizontal curve used..

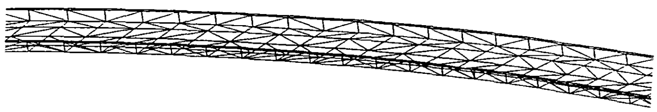

FIGURE 1 Isometric plot of typical 3-D computer model.

For each horizontal alignment four different vertical alignments were developed to correspond to A values of 8, 10, 12, and 14. Each vertical alignment contained an approach grade of 6 percent and descending grade that varied from 2 to 8 percent. The minimum vertical curve length was calculated by using the minimum K value for the appropriate design speed.

Each model was generated with x, y, and z coordinate points along the ridgeline established by the cross-sectional template. These ridgelines were located at the center of the travel lane, at the outside edge of the travel lane, at the edge of the shoulder or face of the barrier, and at the two top corners of the barrier, which are located 863.60 mm (34 in.) above the roadway surface. Transverse ridgelines were constructed at 3.05-m (10-ft) intervals starting at station 10+00 and continuing until the end of the vertical alignment. The 3.05-m (10-ft) interval was chosen to describe the connector as accurately as possible without consuming excessive amounts of computer memory. Figure 1 shows an isometric plot of a typical 3-D computer-generated wire-frame model developed for the study.

Sight Distance Evaluation

The location of the driver's eye and object was based on the location of the vertical curve. The purpose was to place the driver's eye and object location inside both the vertical and horizontal curves. Since the vertical alignments were designed with symmetric parabolic curves, the driver's eye and object were placed equal distances from the vertical PI station.

Figure 2 is a 3-D plot of a connector from the perspective of the driver. The computer camera position was located at the driv-

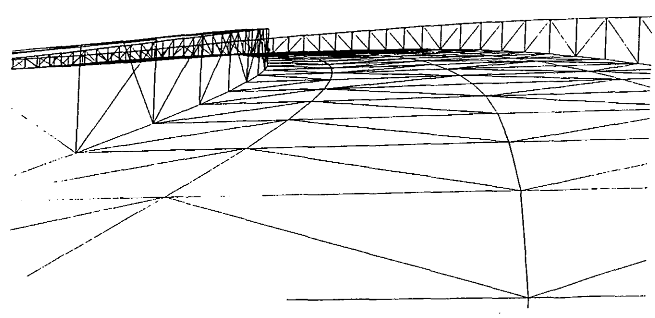

FIGURE 2 3-D plot of driver's perspective view.

er's eye height of 1.07 m (3.5 ft) above the road surface and was directed along the line of sight toward the 152.4-mm (6-in.)-high object location. The expectation from this perspective view was that the critical point along the line of sight would be a point where the barrier and the pavement surfaces meet. This would indicate that both the horizontal and vertical alignments are the design controls for that particular combination of *K*, *A*, *R*, and *M*. What became obvious after reviewing all the models from the driver's perspective was that the line of sight was obstructed only by the barrier and not the roadway surface. The controlling feature thus became the barrier and not the roadway surface. This would indicate that when the cross slope is also taken into consideration, the horizontal alignment becomes the controlling geometric feature for all combinations of minimum horizontal and vertical curvatures.

Another interesting observation from these perspective views was that the roadway surface was visible from beginning to end along the line of sight. This was due to the influence of a favorable cross slope that increased the length of roadway surface visible to the driver because it both lowered the roadway surface elevation along the line of sight and tilted the roadway surface so that it faced the driver. Again what was expected was that a portion of the roadway surface would not be visible to the driver. From a traditional 2-D plot of a vertical alignment (profile view), a portion of the pavement is not visible to the driver because the end of the line of sight is 152.4 mm (6 in.) above the roadway surface. To confirm this situation two separate graphs were developed and compared. The top plot in Figure 3 was developed by plotting surface elevation points along the center of the travel lane (the horizontal curve) from the point of the driver's eye to the object location. The results indicate a 2-D relationship in which a portion of the roadway surface is not visible to the driver. The bottom plot in Figure 3 was developed by plotting the surface elevation points along the line of sight. The plot clearly shows that the line of sight is not obstructed by the roadway surface.

The question that needs to be answered is, What is the minimum vertical curve length that produces the situation in which the line of sight is also obstructed by the roadway surface? To answer this, the study investigated less than minimum vertical curve lengths in which the line of sight intersects the point where the barrier meets the roadway surface. This would produce vertical curve lengths less than the minimum recommended by AASHTO.

The procedure for determining the minimum 3-D vertical curve length consisted of making incremental reductions in the minimum 2-D vertical curve length and then plotting the surface elevation along the line of sight, in addition to plotting a line representing the line of sight. The vertical curve length that produced the situation in which the roadway surface blocked the line of sight was considered the minimum 3-D vertical curve length. Figure 4 is a series of graphs that show plots of the surface elevations along the lines of sight (solid lines) and lines representing the lines of sight (dashed lines) for different values of vertical curve length. This graphical procedure was repeated for all combinations of *K*, *R*, and *M*. For clarification the absolute minimum vertical curve length is described as the 3-D vertical curve length, whereas the minimum vertical curve length required by *K* values is described as the 2-D vertical curve length.

The results of this procedure are listed in Table 2 for a design speed of 72.42 km/hr (45 mph). Table 2 lists the 2-D vertical curve values, 3-D vertical curve values for a cross slope of 0.08, and 3-D vertical curve values for a cross slope of 0.06. The results indicate that the cross slope and horizontal offset (*M* value) have a significant effect on the absolute minimum vertical curve length. They also indicate that the 3-D vertical curve lengths are significantly smaller than the conventional 2-D vertical curve lengths. For example, a cross slope of 0.08 and an *M* value of 2.44 m (8 ft) produces 3-D vertical curve lengths approximately 28 percent less than the 2-D vertical curve lengths for a design speed of 72.42 km/hr (45 mph) and all values of *A*. In addition for a cross slope of 0.08 and an *M* value of 5.49 m (18 ft), the 3-D vertical curve length is approximately 46 percent less than the required 2-D vertical curve length for a design speed of 72.42 km/hr (45 mph). The results for a cross slope of 0.06 were slightly less than the results for a cross slope of 0.08.

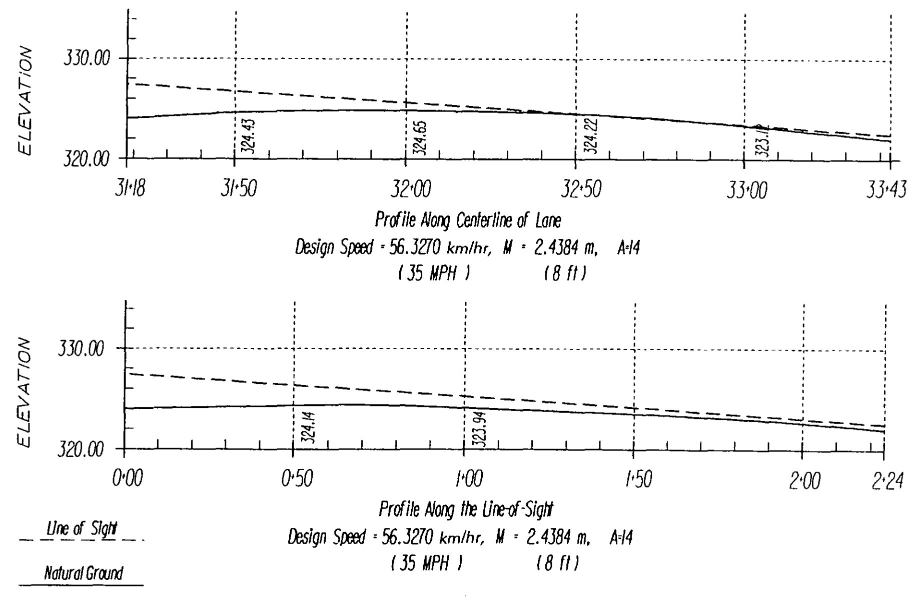

FIGURE 3 Profile plots.

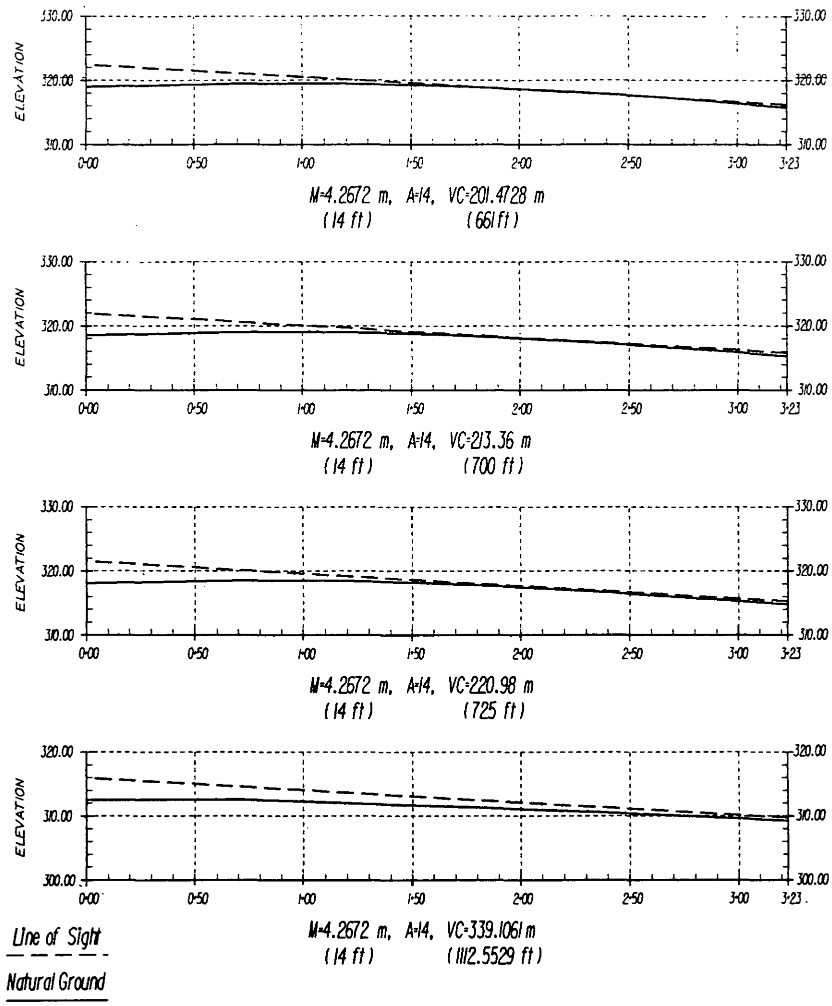

FIGURE 4 Profile plots with incremental reductions in vertical curve lengths.

3-D Measurement of SSD

The purpose of making a 3-D measurement of SSD was to determine if there is a significant difference in the current procedure for determining SSD measured in the 2-D horizontal plane (x and y coordinates only) and the actual 3-D distance between the driver's eye and the object. The conventional approach to determining the available SSD is to consider the horizontal and vertical geometries separately. The horizontal SSD is determined from the 2-D plan view. This means that only the horizontal alignment (x and y coordinates) is considered. The vertical SSD is determined from the profile view, in which only the station and elevation (x and y coordinates) are considered. The approach in this paper for measuring 3-D SSD consisted of calculating x, y, and z coordinates along the alignment (center of the travel lane) in increments of 0.08 m (0.25 ft). For this procedure a large number of coordinate points were developed to increase the accuracy of measurement. For example the 72.42-km/hr (45-mph) design speed criteria requires an SSD value of 99.06 m (325 ft). In increments of 0.08 m (0.25 ft), this results in 1,300 individual coordinate points that were used to calculate the 3-D distance. A spreadsheet was used to sum the distance between each point. This result was then compared with the horizontal measure of SSD that only considers the x and y coordinates. Table 3, which lists the results, indicates that the difference between 2-D and 3-D SSDs is very small. The largest difference was only 29.96 mm (0.0983 ft). What must be noted is that the 3-D measurement occurred in the area of the vertical curve and not on a vertical tangent grade. This situation minimizes the difference in 2-D and 3-D measurements. Another interesting observation is that as the vertical curve length increases, the difference between the 2-D and the 3-D measurements decreases. This is because the driver's eye and object were located within the vertical curve and the elevation difference between each point decreases as the vertical curve length increases.

The differences in 2-D and 3-D measurements were also examined on a vertical tangent grade. Measurement on a vertical tangent grade produced the most significant difference in SSD because of the lengths of roadway surface measured in the 2-D and 3-D environments. The 3-D approach to measurement consisted of measuring the actual roadway surface along the center of the travel lane, whereas the 2-D approach measured the length of roadway projected on a horizontal plane. For this analysis a range of grades from 2 to 8 percent was chosen because they presented the most commonly used grades in the design of a road-

TABLE 2 Values of 3-D Vertical Curve Lengths

Algebraic Difference (%)	M (m)	2-D Min. VC (m)	X-Slope = 0.08 3-D Min. VC (m)	X-Slope = 0.08 % Decrease	X-Slope = 0.06 3-D Min. VC (m)	X-Slope = 0.06 % Decrease
A=14	2.44	339.11	244.14	28.0	262.44	22.6
	3.05	339.11	228.30	32.7	248.41	26.7
	4.27	339.11	201.47	40.6	224.03	33.9
	5.49	339.11	180.14	46.9	203.61	40.0
A=12	2.44	290.66	209.70	27.9	225.25	22.5
	3.05	290.66	195.99	32.6	213.06	26.7
	4.27	290.66	173.13	40.4	192.33	33.8
	5.49	290.66	155.14	46.6	175.26	39.7
A=10	2.44	242.20	174.96	27.8	187.76	22.5
	3.05	242.20	163.37	32.5	178.00	26.5
	4.27	242.20	144.78	40.2	160.93	33.6
	5.49	242.20	129.84	46.4	146.61	39.5
A=8	2.44	193.77	139.90	27.8	150.57	22.3
	3.05	193.77	131.06	32.4	142.65	26.4
	4.27	193.77	116.13	40.1	128.93	33.5
	5.49	193.77	104.24	46.2	117.96	39.1

way. Table 4 shows the results for design speeds of 40.23 km/hr (25 mph), 56.32 km/hr (35 mph), and 72.42 km/hr (45 mph). The results also revealed that as the grade increased the difference in the 2-D and the 3-D measurements also increased. These results, when compared with those in Table 3, were significantly higher for tangent grades of greater than 5 percent. The difference in comparison with the overall value of SSD was extremely small, however.

CONCLUSIONS AND RECOMMENDATIONS

Summary

The results of the graphical procedure demonstrated that the roadway feature blocking the line of sight was the longitudinal barrier for all 48 computer-generated 3-D models. This indicated that the horizontal alignment (radius and offset distances) was the con-

TABLE 3 3-D Measurement of SSD on Crest Vertical Curve

Algebraic Difference (%)	40.23 km/h Design Speed (25 mph) M (m)	SSD (m)	3-D Measure (m)	56.32 km/h Design Speed (35 mph) M (m)	SSD (m)	3-D Measure (m)	72.42 km/h Design Speed (45 mph) M (m)	SSD (m)	3-D Measure (m)
8	2.44	45.72	45.75	2.44	68.58	68.60	2.44	99.06	99.09
10	2.44	45.72	45.74	2.44	68.58	68.59	2.44	99.06	99.07
12	2.44	45.72	45.74	2.44	68.58	68.59	2.44	99.06	99.07
14	2.44	45.72	45.74	2.44	68.58	68.59	2.44	99.06	99.07
8	3.05	45.72	45.75	3.05	68.58	68.60	3.05	99.06	99.09
10	3.05	45.72	45.74	3.05	68.58	68.59	3.05	99.06	99.07
12	3.05	45.72	45.74	3.05	68.58	68.59	3.05	99.06	99.07
14	3.05	45.72	45.74	3.05	68.58	68.59	3.05	99.06	99.07
8	4.27	45.72	45.75	4.27	68.58	68.60	4.27	99.06	99.09
10	4.27	45.72	45.74	4.27	68.58	68.59	4.27	99.06	99.07
12	4.27	45.72	45.74	4.27	68.58	68.59	4.27	99.06	99.07
14	4.27	45.72	45.74	4.27	68.58	68.59	4.27	99.06	99.07
8	4.96	45.72	45.75	5.49	68.58	68.60	5.49	99.06	99.09
10	4.96	45.72	45.74	5.49	68.58	68.59	5.49	99.06	99.07
12	4.96	45.72	45.74	5.49	68.58	68.59	5.49	99.06	99.07
14	4.96	45.72	45.74	5.49	68.58	68.59	5.49	99.06	99.07

TABLE 4 3-D Measurement of SSD on Vertical Tangent Grade

Tangent Grade (%)	40.23 km/h Design Speed (25 mph) SSD (m)	Actual Distance (m)	56.32 km/h Design Speed (35 mph) SSD (m)	Actual Distance (m)	72.42 km/h Design Speed (45 mph) SSD (m)	Actual Distance (m)
2	45.72	45.73	68.58	68.59	99.06	99.08
3	45.72	45.74	68.58	68.61	99.06	99.11
4	45.72	45.76	68.58	68.63	99.06	99.14
5	45.72	45.78	68.58	68.67	99.06	99.18
6	45.72	45.80	68.58	68.70	99.06	99.24
7	45.72	45.83	68.58	68.75	99.06	99.30
8	45.72	45.87	68.58	68.80	99.06	99.38

trolling geometric feature of the interchange connectors, even though the vertical alignment was designed with the same design speed criteria. In general, when evaluating geometric combinations that contain severe horizontal and crest vertical curve combinations, the horizontal alignment of the connector will probably be the controlling geometric feature. If this is the case, as long as the vertical alignment is designed with K values equal to or greater than the design speed of the horizontal alignment, the horizontal feature will control the available SSD. These results also demonstrate the importance of determining the available SSD for both the horizontal and the vertical alignments before concluding the design speed of the connector.

As previously stated the current design standards were developed by assuming a 152.4-mm (6-in.) object height and a 1.07-m (3.5-ft) driver eye height. On the basis of this definition, AASHTO developed a methodology for calculating minimum crest vertical curve lengths that uses the roadway surface as the feature that obstructs the line of sight. This methodology considers sight distance as the only criterion for determining minimum crest vertical curve length. The methodology was developed by using 2-D criteria (considering only the elevation and distance along the center of the travel lane), which limits the application to locations where a crest vertical curve is combined with a horizontal curve that does not require a cross slope. In actual design this situation is only applicable to a combination of a crest vertical curve with a straight horizontal alignment.

The results of the present study clearly indicate that when a crest vertical curve is combined with a horizontal curve that requires a cross slope, the roadway surface does not block the line of sight. Significant reductions in vertical curve lengths are possible when viewed from the perspective of only providing the minimum amount of sight distance. Reductions as small as 28 percent for M = 2.44 m (8 ft) and as large as 46.2 percent for M = 5.49 m (18 ft) are possible for a design speed of 72.42 km/hr (45 mph). These results, if used in design, would violate the vertical alignment design standards recommended by AASHTO. However according to the procedure for measurement described by AASHTO, the results would not violate SSD guidelines because the 152.4-mm (6-in.) object would continue to be visible to the driver.

By using the current 2-D methodology for determining minimum crest vertical curve lengths, the results also indicated that the design of a crest vertical curve when combined with a horizontal curve that requires a cross slope is more conservative because of the additional available vertical sight distance. For the models developed in the present study, in which the minimum vertical curvature is combined with the minimum horizontal curvature, the additional vertical sight distance was limited because of the lateral obstruction. This would indicate that the design of a roadway with a higher horizontal design speed than a vertical design speed will result in a measurable vertical sight distance that is longer than the sight distance calculated by using the 2-D vertical design equations. The additional available vertical sight distance constitutes an amount not realized in the current crest vertical curve methodology.

Recommendations

The results indicate that significant reductions in vertical curvature are possible if one is designing a roadway solely on the basis of providing the minimum amount of sight distance. However a design made solely from this perspective may lead to other unexpected problems. Design consistency would be one potential problem area because each geometric combination would produce unique driving conditions. Drivers would be required to drive solely on their visual capabilities. They would also be required to expect a sharper crest vertical curve when a cross slope is introduced.

These new lower crest vertical curve lengths should also be investigated from the perspective of driver discomfort. Caution should be used in incorporating the lower values because they may produce an unacceptable level of driver discomfort. It is strongly recommended that designers use values that are equal to or exceed AASHTO's lower-range values for crest vertical curve lengths. According to the Texas design manual, "Greater than minimum SSD should normally be used and minimum values used only in select instances where economic or other restrictive conditions dictate" (*9*). The point of using greater than minimum values by reclassifying AASHTO's lower-range values as minimum values and the upper-range values as desirable values is also emphasized. The use of values equal to or greater than the minimum would also support AASHTO's recommendation of designing with "prevalent expectancies" because it is one of the most important ways to aid driver performance. AASHTO states that when drivers "do not get what they expect, or get what they do not expect, errors may result" (*1*). Additional research is needed to evaluate all of the ramifications associated with using values of crest vertical curvature less than the recommended minimum.

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Geometrically Induced Roughness at Grade Breaks

KAM K. MOVASSAGHI, JIM LEE, AND AMIT KUMAT

Grade breaks are used to make the transition from a secondary roadway elevation to the elevation of the main highway. At the design stage, wrong decisions regarding the geometrical composition of the transition profile may produce an undesirable level of roughness that may be unsafe. Although a variety of instruments can be used to physically measure the road roughness, there are no known standards for analyzing and estimating the level of profile roughness during the design stage. A methodology that uses the International Roughness Index (IRI) as the performance measure of profile roughness is presented. A series of simulation experiments was performed for six types of intersections to examine the relationship between roughness and the intersection design parameters. The results of the experiments show that profile roughness is affected by the transition curve parameters as well as the elevation difference between the main highway and the intersecting secondary roadway. A decision support system called SIDRA (System for Intersection Design with Roughness Analysis) was developed to estimate the roughness of an existing profile or to generate an alternate profile with a lower level of roughness. Statistical analysis showed a close correlation between SIDRA-generated results and data measured in the field. SIDRA-produced values of IRI were also correlated with the serviceability index, a commonly accepted roughness measure.

Design and management of highway networks have been going through technologically driven changes to provide the best possible service to highway users. The advent of computer visualization tools such as computer-aided design and drafting (CADD) and geographic information systems (GISs) allows designers to visually experience highway geometries before construction. These tools are used by the designers to achieve a basic design objective: to produce the best possible plan to meet the specified needs. In designing a transition profile, although one can easily view it at the design stage, at the present time, there are no available means of determining its inherent roughness owing to design assumptions.

Geometric design of a highway intersection usually incorporates one or more grade breaks to allow a transition from a secondary roadway elevation to the elevation of an intersecting main highway. The design of a grade break is affected by factors such as horizontal alignments, profile, plan of the intersection, clearances, and horizontal dimensions of the highway cross sections (*1*). An improperly designed grade break will produce a level of roughness that may not be acceptable to highway users or that may possibly be unsafe. Currently AASHTO does not have specific guidelines for the design of grade breaks. A search of the literature on established methodologies produced no results. It is conceivable that there may be design procedures at some departments of transportation or local municipalities for their internal use; however, the authors are unaware of such processes. To the best of the authors' knowledge, there is no tool that can be used to analytically generate a measure of roughness at the intersection, produce a profile with a low level of roughness, or improve an existing design.

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Road roughness is defined as the variation in surface elevation that induces vibrations in traversing vehicles (*2*). By causing vehicle vibration, roughness has a direct influence on vehicle wear, ride comfort, and safety (*3,4*). Road roughness is gaining increasing importance as an indicator of road condition, both in terms of pavement performance and as a major determinant of road user costs. Of the various kinds of desired surface qualities, in the public view, road roughness has a strong influence on the measure of serviceability. In the AASHTO Road Test, road roughness was found to be the primary correlate of the present serviceability index (SI) (*5*). As a result many state highway departments and transportation agencies use road roughness to estimate the SI.

The main objective of the study was to develop a decision support system for the design of a highway intersection with an acceptable level of roughness. The performance measure of the design is based on the value of the International Roughness Index (IRI). IRI calculation is accomplished by incrementally computing four variables along the profile. These four variables—defined in a previous report (*6*) in which a detailed description of IRI is presented—simulate the dynamic response of a reference vehicle traveling over the profile.

The present study considered six different profiles ranging from a simple T-intersection to a compound design that included several vertical curves. The following section presents the particular makeup of each profile.

TRANSITION PROFILES

To define the variables associated with a particular profile, consider Figure 1. This type of profile, joining a secondary roadway and the main highway at different elevations, is referred to as profile SUD (symmetrical up-down); it is symmetrical about the centerline of the main highway (Point B) and consists of a combination of a parabola (P), a tangent (T), and a sag (S) curve. In this paper, for the sake of convenience, the term *parabola*—in contrast to the term *sag*—is used to designate the AASHTO type II crest vertical curve (*1*). For all the analyses, the starting profile consists of a 10.8-m (36-ft) tangent length of 0 percent gradient along the secondary roadway joined to a sag curve of 45 m (150 ft), which is joined by a second tangent of 51 m (170 ft) and finally a parabola of 45 m (150 ft) that joins the tangent to the cross slope of the main highway. The width of the main highway is assumed to be 14.4 m (48 ft), with a cross slope of 2.5 percent on both sides of the centerline. The difference in elevation be-

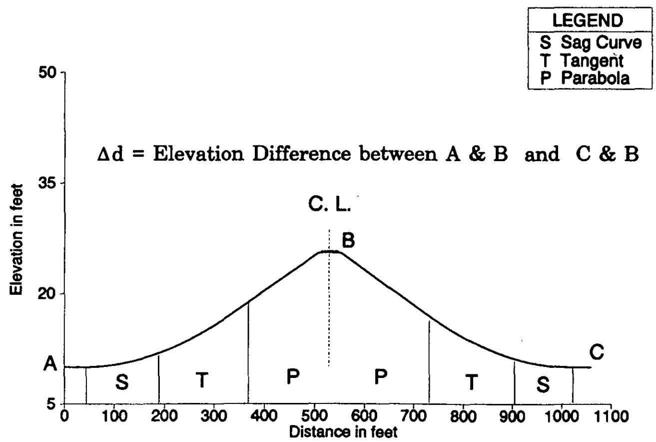

FIGURE 1 SUD profile.

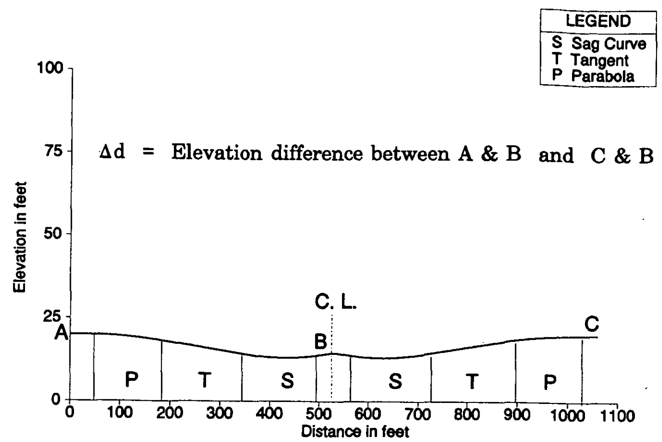

FIGURE 3 Profile SDU.

tween the starting point on the secondary roadway (Point A in Figure 1) and the elevation of the crown of the main highway (Point B) is defined as the elevation difference and is designated as Δd.

The properties of the three curves (P, T, and S) are controlled by three parameters: start gradient, end gradient, and the length of the curve. For a given set of values and an elevation difference, the calculated value of IRI for the profile is the performance measure of that profile. The following sections present the descriptions of the remaining five profiles.

Profile TINT

Profile TINT is the simplest type of intersection considered in the study. This profile played an important role in the study because the S-T-P combination was used as a building block in more complex transitions. Figure 2 shows a TINT profile.

Profile SDU

When the right side of the main highway is a mirror image of the left side of the main highway, the profile is called a *symmetrical profile*. The SDU profile has a P-T-S combination on the left side of the main highway and an S-T-P combination on the right side. Figure 3 shows an SDU profile.

Profile XUU

For asymmetrical profile XUU, the elevation difference of the secondary roadway and the main highway is kept constant on both sides of the main highway. Both sides of the main highway have an S-T-P combination (Figure 4).

Profiles SVV and SCC

A compound profile consists of more than two tangents on one side of the main highway. There are two types of compound profiles: concave (C) and convex (V). A concave profile consists of a sag curve, a tangent, a parabola, a tangent, and a sag curve. In a concave profile, the roadway goes upward and then downward. The convex profile consists of a parabola, a tangent, a sag curve, a tangent, and a parabola. In a convex profile the roadway goes

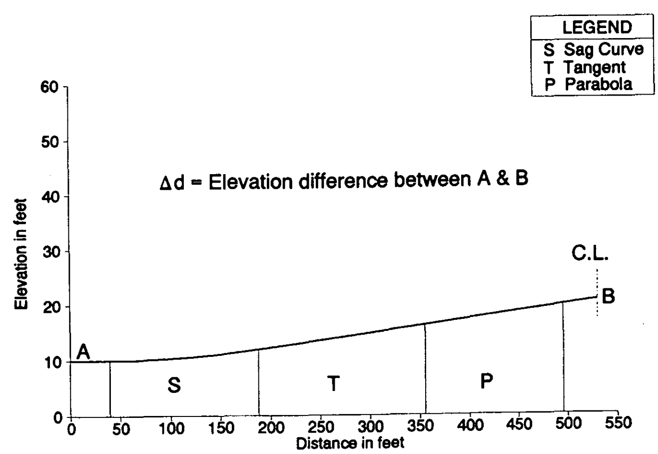

FIGURE 2 Profile TINT.

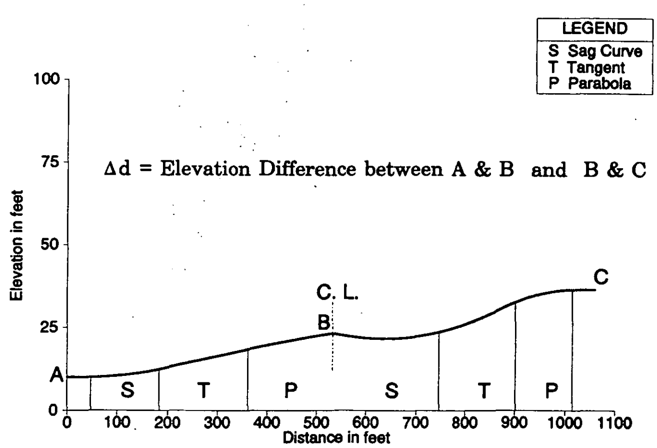

FIGURE 4 Profile XUU.

downward and then upward. Profile SVV is convex on both sides of the main highway (Figure 5), and profile SCC is concave on both sides of the main highway (Figure 6).

APPROACH

To examine the relationship between roughness and design parameters for intersections at grade, a series of simulation experiments was performed on each of the six profiles discussed in the previous sections. The experiments were performed by varying the parameters of two curves for three values of elevation difference between the secondary roadway and the main highway to generate profiles. For every profile generated two corresponding IRI values were calculated, one for each direction along the profile. This procedure was repeated for all six intersections.

On the basis of the results from the simulation experiments, a decision support system called *SIDRA* (System for Intersection Design with Roughness Analysis) was developed. When provided with the values of curve parameters for an existing intersection, a highway designer can use SIDRA to generate alternative or improved designs by varying the curve parameters within the feasible ranges.

The validation process of SIDRA consisted of comparing the generated IRI values with the measured IRI values on four existing intersections in Baton Rouge, Louisiana. The measured values were obtained by using two road roughness measuring devices: the K. J. Law model 8300 Roughness Surveyor and the Face Dipstick. The comparison of the IRI values is presented later in the paper.

The SI has commonly been used as a measure of riding quality on roadways in many places. The relationship of SI to IRI for roadways has been examined by many researchers. However there are no reported data on the relationship between SI and IRI at intersections. A correlation study was performed on the basis of the values of IRI and SI for 10 intersections in Baton Rouge, Louisiana.

SIMULATION PROCEDURE

This section presents the simulation procedure performed on six types of intersections. Each of the six profiles is characterized by

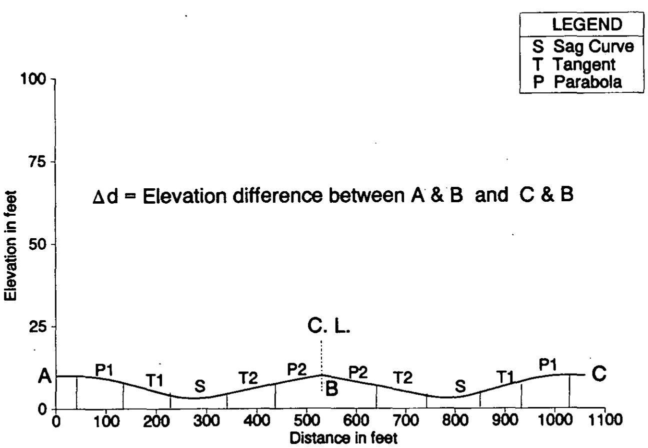

FIGURE 5 Profile SVV.

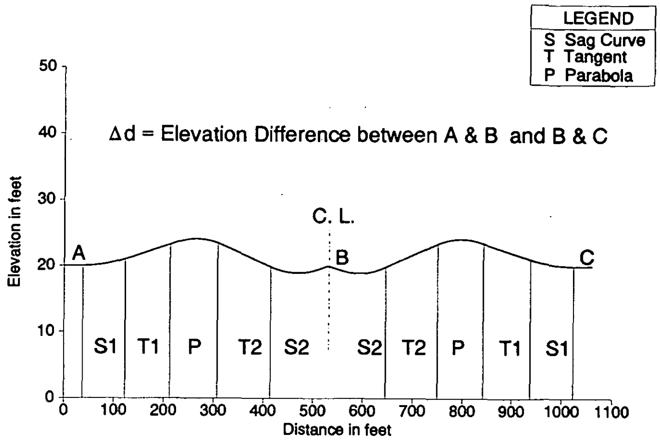

FIGURE 6 Profile SCC.

the number of curves that make up the profile and a set of curve parameters. In the simulation profiles are generated systematically by changing one parameter at a time and adjusting the other parameters to meet the specified elevation difference. On the basis of the values of the curve parameters, a set of elevations is calculated for the profile. The elevations are generated at 1-ft intervals, and a cumulative IRI value is computed for the profiles in both directions; these profiles are called *Final-IRI* and the *opposite Final-IRI*. The last computed IRI value at the end of a profile is the cumulative IRI for the total length of the profile and is referred to as the *Final-IRI*. The opposite Final-IRI is determined by traversing the profile in the opposite direction. The general simulation procedure is summarized below.

1. *Basic design of curve combination.* For a given type of intersection, preliminary studies are conducted to determine a curve combination that provides a smooth transition from a secondary roadway to the main highway. Normally at least one tangent is used to provide a smooth transition between curves. The number of curves including the tangent(s) in the transition profile is called *N*.
2. *Initial setting of design parameters.* An appropriate initial value for each of the following design parameters is selected: the total horizontal length of the intersection, the width and the cross slope of the main highway, the initial gradient of the secondary roadway, and the elevation difference between the secondary roadway and the main highway.
3. *Initial setting of curve parameters.* On the basis of an initial setting of design parameters in Step 2, the curve parameters of a feasible profile (as defined by AASHTO geometric design standards) are determined by setting the lengths of the three curves approximately equal. Next, a profile based on the curve parameters is generated and the Final-IRI and opposite Final-IRI are computed. Subsequently, the curve parameters and the other design parameters to be varied in the simulation experiments are identified.
4. *Experiments with varied curve lengths.* Given an *N*-curve combination, a number of feasible profiles for intersection design are generated in the following way.
   a. Increase the gradient of the first tangent in the *N*-curve combination by a small percentage, *R*. Vary the length of two other curves to accommodate this change. Generate a profile

on the basis of the new curve parameters and compute the Final-IRI and opposite Final-IRI.

b. Repeat Step *a* above by increasing the tangent *R* percent and adjusting the length of the two curves selected in Step *a* above to generate a feasible profile. This step is repeated until a feasible profile cannot be generated either by increasing the tangent gradient or by exhausting all possible tangent lengths.

c. For all the feasible profiles generated in Steps *a* and *b*, select the profiles with the lowest Final-IRI and the lowest opposite Final-IRI. Output the curve parameters of the two profiles.

d. Repeat Steps *a*, *b*, and *c* by varying the lengths of the two other curves. For an *N*-curve combination, this step is repeated $C(N,2)$ times, as defined below.

$$C(N,2) = \frac{N!}{(N-2)!2!}$$

5. *Experiments with varied elevation differences.* The elevation difference between the secondary roadway and the main highway is increased by a small percentage, *E*. Steps 3 and 4 are repeated with the new elevation difference.

In most of the experiments completed in the present study, the elevation difference is tested at three different levels. Therefore $3 \times C(N,2)$ sets of experiments are performed for each of the six types of intersections. The number of profiles to be generated per experimental set is dependent on the initial settings of the design parameters and the step size *R*. An illustrative example is presented below to demonstrate the simulation procedure as applied to profile TINT.

SIMULATION EXAMPLE: PROFILE TINT

Preliminary studies on T-intersections indicated that both the curve parameters and the elevation difference between the main highway and the secondary roadway affect the maximum IRI and the Final-IRI. Figure 2 depicts a TINT profile that has a total horizontal length of 159 m (530 ft). The profile starts with a 0 percent tangent that is 10.8 m (36 ft) long on the secondary roadway. The tangent is then joined by a combination of a 45-m (150-ft) sag curve, a 51-m (170-ft) tangent, and a 45-m (150-ft) parabola that connects to the main highway with a width of 14.4 m (48 ft) and a cross slope of 2.5 percent.

Experiments are performed on profile TINT by systematically varying two factors: (a) the lengths of two of the three curves in the S-T-P combination and (b) the elevation difference between the main highway and the secondary roadway. Possible pairs of curves from the S-T-P combination (S-T, P-T, and S-P) are tested at three levels of elevation differences. The levels are selected in a manner that grade changes of approximately 1, 2, and 3 percent would result in the intersection.

The experiments can be divided into three groups, as shown in Table 1. For example Group TINT.ST experiments are performed by reducing the tangent length and increasing the sag length to meet the induced 0.02 percent change in the tangent gradient. In each group experiments are repeated by using the same procedure for three different elevation differences, that is, 1.59 m (5.3 ft), 3.18 m (10.6 ft), and 4.77 m (15.9 ft). Two profiles with the lowest Final-IRI and opposite Final-IRI are selected from each experiment for a total of 18 profiles. The curve parameters of the 18 profiles and their Final-IRI or opposite Final-IRI are summarized in Table 2. Figure 7 shows the IRI variation along the TINT profile for one experiment. The variation of IRI as a function of the length of parabola is plotted in Figure 8. In Figures 7 and 8, the IRI values are computed for both directions of travel. The arrows in the figures show the direction of travel.

TABLE 1 TINT Experiment Groups

Experiment Group	Sag Length	Tangent Length	Parabola Length
TINT.ST	Increase	Decrease	Constant
TINT.PT	Constant	Decrease	Increase
TINT.SP	Increase	Constant	Decrease

SIMULATION RESULTS

A large number of simulation experiments were performed in the study for six types of intersections on the basis of the simulation procedure described in the previous section. The purpose of the simulation is to study the relationship between design parameters and the roughness. In general, the Final-IRI value is directly proportional to the elevation difference between the secondary roadway and the main highway. IRI variation is also sensitive to the variation in the parabola length. The suggested parameter settings from the simulation experiments are summarized in Table 3 by profile type.

SOFTWARE IMPLEMENTATION

A decision support system called SIDRA was developed on the basis of the experimental results. It is written in QuickBASIC

TABLE 2 Results of TINT Experiments

Experiment Type	Sag Length (ft.)[a]	Tangent Gradient %	Tangent Length (ft.)	Parabola Length (ft.)	Final-IRI (in/mile)[b]	Opposite Final-IRI (in/mile)
			Δd = 5.3 ft.			
ST	319	1.24	1	150	2.7	-
ST	319	1.24	1	150	-	13.83
PT	150	.9	170	150	5.8	-
PT	150	.9	170	150	-	17.01
SP	150	.9	170	150	5.8	-
SP	150	.9	170	150	-	17.01
			Δd = 10.6 ft.			
ST	290	3.29	30	150	4.73	-
ST	293	3.31	27	150	-	13.69
PT	150	2.59	31	289	6.89	-
PT	150	2.55	170	150	-	16.31
SP	196	2.75	170	104	6.36	-
SP	253	2.97	170	47	-	14.49
			Δd = 15.9 ft.			
ST	289	5.4	31	150	11.18	-
ST	289	5.4	31	150	-	16.76
PT	150	4.7	32	288	11.97	-
PT	150	4.7	32	288	-	20.1
SP	174	4.32	170	126	14.47	-
SP	174	4.32	170	126	-	19.52

[a]1 ft. = 0.3 m; [b]1 in/mile = 15.875 mm/km

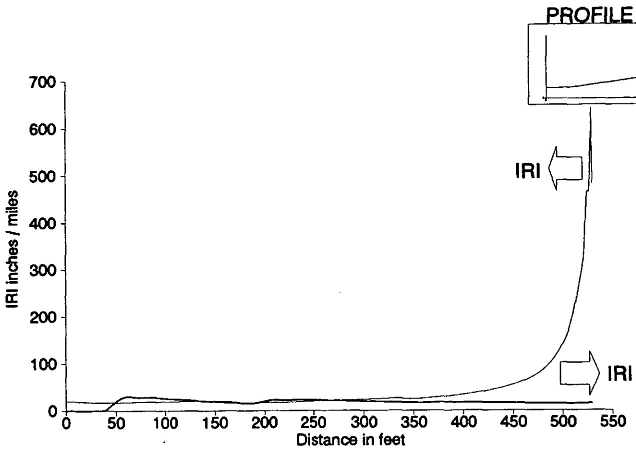

FIGURE 7 IRI variation for TINT profile.

TABLE 3 Summary of Simulation Results

Profile	Basic Design	Parameter Settings for Low IRI Values (approximate values)		
		Curve Type	Curve Length as % of Profile Length	IRI is Sensitive to the Length of
TINT	S-T-P	T P	6 9	P
SUD	S-T-P (left) P-T-S (right)	T P	12 4	
SDU	P-T-S (left) S-T-P (right)	T P	12 50	S and P
XUU	S-T-P (for both sides)	T P	12 50	P
SVV	P1-T1-S-T2-P2 (for both sides)	T1 P1 T2 P2	12 22 16 4	P2
SCC	S1-T1-P-T2-S2 (for both sides)	T1 S1 T2	12 24 34	

language (*7*) on an IBM compatible PC. The user has the option of calculating the roughness of a profile when the elevation points are provided in an ASCII file. When provided with the elevation difference and the length between the secondary roadway and the main highway, the program generates feasible profiles and selects the ones with lower IRI values. There is a separate module for each type of intersection.

VALIDATION OF SIDRA

To validate the accuracy of SIDRA surface elevation data were collected from two street intersections in Baton Rouge, Louisiana. On the basis of those data IRI values were generated by SIDRA and were compared with the values obtained from two road roughness measuring devices: the K. J. Law model 8300 Roughness Surveyor and the Face Dipstick. The K. J. Law surveyor uses an ultrasonic road sensor and an accelerometer to measure the longitudinal profile of the road. The measured profile is used to compute the Mays Index, which is identical to the IRI (*8*). The Face Dipstick measures the profile by automatically recording the change in elevation at every foot. An on-board computer system was used to compute the IRI values of the profiles.

Two sets of data were obtained for each intersection: one for each traffic lane. The road test was conducted by the technical staff of the Louisiana Transportation Research Center (LTRC). Table 4 depicts the IRI values. Close correlations between the values were observed.

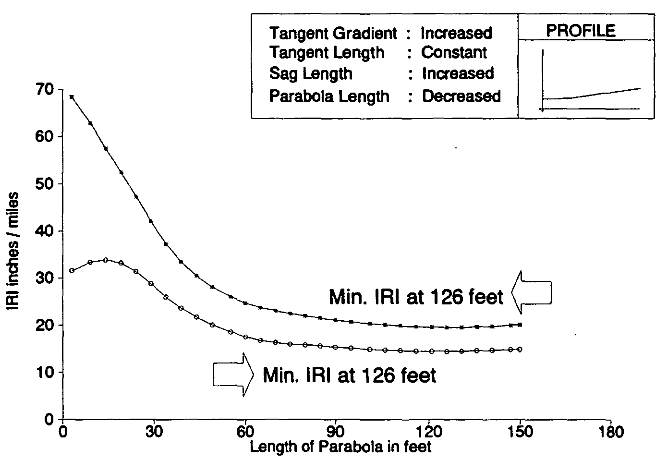

FIGURE 8 IRI variation with respect to length of parabola.

RELATIONSHIP OF IRI TO SI

The concept of SI was first introduced at the AASHTO Road Test in the late 1950s. The basic concept is that a panel of users rates the pavement as to its roughness and ability to serve the motoring public. The SI scale is from 0 to 5, with a pavement rated as 0 being impassable and a pavement rated as 5 being perfectly smooth. On the other hand, an IRI value is the measure of roadway roughness; in contrast to SI, it does not clearly express the level of rider's comfort. Since the AASHTO Road Test, the SIs of the roads have been routinely correlated with the outputs of various roughness measuring devices. In one such study, the relationship of SI to IRI (as measured with the K. J. Law model 8300 Roughness Surveyor) has been documented by Cumbaa (*8*). The study concluded, "The International Roughness Index appears to be a useful tool for identification of the relative roughness levels between pavements and for predicting the rideability rating which a panel might provide irrespective of pavement type."

To the best of the authors' knowledge, the SI concept has never been applied to the geometric design of intersections. Furthermore the propriety of extending the established relationships to inter-

TABLE 4 IRI Comparison with Instrument Readings

Hwy Name/ Lane	K.J.Law IRI (in/mile)*	Dipstick IRI (in/mile)*	SIDRA IRI (in/mile)*	K.J.Law / SIDRA	Dipstick / SIDRA
LA-1/N	147	141.43	140.9	1.040	1.003
LA-1/S	210	208.73	208.8	1.005	0.999
PICOU/E	278	261.18	262.7	1.050	0.994
PICOU/W	278	267.08	266.5	1.040	1.002

*1 in/mile = 15.875 mm/km

sections for which a panel of motorists may rate intersection roughness differently than normal highway roughness has not been verified. A section of highway and an intersection may have the same measured roughness over a given length, but a panel may subjectively rate them as having differing SIs.

Ten intersections in Baton Rouge, Louisiana, were rated by an experienced team of LTRC staff while taking measurements of IRI with the K. J. Law model 8300 Roughness Surveyor. Table 5 summarizes the data for the 10 intersections. A linear regression was performed on the measured IRI and the rated SI values (Table 5), with IRI as the independent variable. The following relationship resulted:

$$SI = 4.46 - 0.00592 * IRI$$

The coefficient of determination (R^2) of the regression was 0.8, which indicates that there is a good linear relationship between SI and IRI for intersections. This regression equation was used to estimate the SI values for the measured IRI values at the 10 intersections. The estimated SI (produced by the regression equation) and the percent difference between the rated and the estimated SIs are presented in Table 5.

To study the relationship between the SIs of roadways and intersections further, the rated SIs of the 10 intersections were plotted against the measured IRI. Figure 9 shows the relationships. A review of Figure 9 indicates that, although there is a strong relationship between IRI and SI, it is possible for a rating panel to subjectively rate the intersection roughness somewhat differently than it rates the highway segments.

The actual IRI of any roadway segment consists of two parts: the IRI due to the design and the IRI due to construction. In the intersection design, one should always strive for a lower value of design IRI, knowing that additional roughness will be added during construction. The LTRC technical staff has recommended that the values given in Table 6 be used as guides for the IRI values of an intersection owing to design.

The IRI of newly constructed highways in Louisiana generally ranges between 1270 mm/km (80 in./mi) and 3175 mm/km (200 in./mi), with a propensity of the value estimated to be approximately 1746 mm/km (110 in./mi). If this latter value is used as the construction IRI and is added to the IRI value owing to the geometric design of the intersection, the sum of the two values can be a good approximation for the total IRI at an intersection.

TABLE 5 Comparison of IRI and SI at Intersections

Intersection	IRI (in/mile)[a]	Rating	Rated SI	Estimated SI	% Difference
Lee Dr. (W.B.) X Highland Road	562	Very Poor	0.5	1.13	126
O'Neal Lane (N.B.) X Florida Blvd.	884	Impassable	0	0	-
Burbank (S.B.) X Ben Hur	230	Fair	2.5	3.09	23.6
Bluebonnet (N.B.) X Highland Road	359	Poor	1.5	2.33	55.3
O'Neal Lane (S.B.) X Old Hammond	276	Fair	2.5	2.82	12.8
Park Blvd. (S.B.) X Tulip	253	Fair	2.5	2.96	18.4
Perkins Rd. (N.B.) X Terrace	180	Good	3.5	3.39	3.1
Hyacinth (W.B.) X Cloverdale	195	Good	3.5	3.30	5.7
Hyacinth (E.B.) X Stuart	160	Very Good	4.5	3.51	22
Burbank (S.B.) X Lee Drive	131	Very Good	4.5	3.68	18.2

[a]1 in/mile = 15.875 mm/km

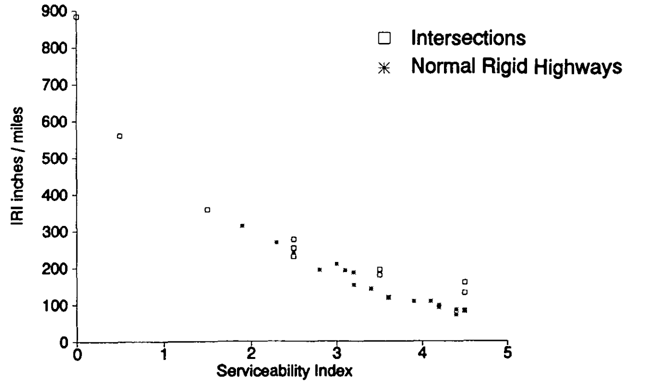

FIGURE 9 Relationship of IRI to SI.

CONCLUSIONS

The objective of the research described here was to develop a decision support system that aids the highway designer in designing an intersection. The project started with a series of simulation experiments to provide an understanding of how various profiles are affected by the properties of the curves that make up the profile. The results showed that the roughness of an intersection is affected both by the curve parameters and the elevation difference between the main highway and the secondary roadway. The values of the curve parameters have a significant effect on the roughness of the road. For most of the intersections, the roughness is directly proportional to the elevation difference between the secondary roadway and the main highway.

From the results of the six sets of simulation experiments, a computer program called SIDRA was developed. SIDRA can generate feasible profiles with low levels of roughness on the basis of the input data, including elevations of the secondary roadway and the main highway, the length between the secondary roadway and the main highway, and the cross slope and the width of the main highway. The results produced by SIDRA are verified in several different ways. The IRI computation is verified by comparing SIDRA output with the IRI measured by two road roughness measuring devices. There was a close match between the generated and measured data.

Finally, a regression model was developed to examine the relationship between IRI and SI. The coefficient of determination indicated a strong relationship between SI and IRI. Overall,

TABLE 6 Recommended Upper Limits for Design

Posted Speed (mph)[a]	SI(overall)	IRI(Design) in/mile[b]
10 to 25	≥ 2.0	190
30 to 40	≥ 2.5	130
45 or greater	≥ 3.0	90

[a]1 mph = 1.6 km/hr; [b]1 in/mile = 15.875 mm/km

SIDRA is an excellent decision support tool that can easily be used at the design stage.

ACKNOWLEDGMENTS

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Cross Sections of High-Occupancy-Vehicle Lanes on Freeways and Arterials

TIMOTHY J. LOMAX AND CHARLES FUHS

The state of the practice related to the design of cross sections for high-occupancy-vehicle (HOV) lanes on freeways and arterial streets is summarized. The summary is based on several documents, including an AASHTO design guide, that have recently examined the operating HOV lane projects in North America to develop dimensions that appear to represent desirable practice and compromises that can operate in constrained situations. The point that HOV operations and design treatments are interrelated and that cross sections cannot be transferred from one project to another without a recognition of the operational improvements that may have been made to allow that cross section to operate is stressed. More freeway HOV projects than arterial street HOV projects are in operation, which provides more certainty regarding the experience on those facilities. The variety of freeway treatments has provided HOV lane designers with the opportunity to analyze the operations on those facilities; as arterial street treatments that allow carpools to use the priority lanes are developed, their design and operating characteristics should be studied to provide information to future projects. The existing guidance is in the form of a set of issues that should be addressed before an arterial street treatment is implemented. Bus-only lanes on streets can provide some guidance to designers, but lanes only for buses and carpools will operate differently, and the design and operating plans should recognize the difference.

There are approximately 45 freeway high-occupancy-vehicle (HOV) lane projects and a varied number of projects that could be defined as arterial street HOV facilities operating in North America. A few of these projects have been operating for more than 15 years, although many have begun service in the last 5 years. It is with this limited experience base (in comparison with the experience base for general-purpose freeway and street facilities) that several efforts have been undertaken to synthesize the lessons learned about designing lanes for HOV projects.

This paper summarizes those efforts in the area of cross-section design for mainlane HOV facilities. It should be noted, however, that the knowledge base continues to grow and should be monitored for changes as the state of the practice evolves.

Another aspect that causes HOV projects to differ from general-purpose facilities is the relationship between design and operation. This is also discussed here. HOV solutions tend to be specific to unique operational and capacity shortcomings and are often applied in highly constrained physical settings. As such projects require a close working relationship among planners, designers, and operators to customize a treatment to the conditions in a corridor. Resulting projects tend to reflect these unique qualities. A design treatment that seems to be successful in one location may not be transferable to another. Many projects have evolved in response to the changing clientele and conditions to which they have been subject. It is important to examine the full context of a design application before using collective experiences to design a new project.

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TYPICAL HOV LANE DESIGN

This paper lists the dimensions suggested by several recent HOV lane design studies and the recent AASHTO publication on design of HOV facilities. There is general agreement on the dimensions that should be used in projects in which major construction or reconstruction will take place. These desirable designs are consistent among the various references. They also illustrate changes made to suit local conditions, such as enforcement agency policies and snow removal requirements.

The dimensions usually listed under the term *reduced* are more varied, but generally reflect a design that has worked well for several agencies over a substantial segment of a corridor. These treatments are frequently applied as an interim step or when a variety of impediments preclude implementation of a desirable HOV lane design.

Most HOV lane projects have been implemented in corridors with six or more general-purpose freeway lanes. If projects are contemplated for four-lane freeways, the resulting project has usually included additional general-purpose lanes, with HOV lane envelopes created for future implementation if demands warrant them. Projects that convert an existing shoulder to an HOV lane are much easier to accomplish if the general-purpose lanes can be narrowed; this process provides a much greater return with three or more directional freeway lanes.

One consistent element in all the HOV lane design guidelines and project characteristics is the need to provide for the safe and effective operation of both the adjacent main lanes and the HOV facility. Lower design standards may adversely affect the performance of the HOV project to provide the travel time and trip reliability improvements that are the selling points of the HOV concept. It is therefore very important for projects to adhere to desirable dimensions and operating characteristics whenever possible. The desirable dimensions presented in this paper typically provide clearance for disabled vehicles to be stored without interfering with HOV lane operation, provide for efficient enforcement operations, and create a perception of a safe and permanent facility.

Projects with dimensions and characteristics that are not only less than the desirable values but that are in some cases less than the values labeled reduced have operated for many years. A common aspect of most of these is a limit on the amount of right-of-way available. These projects are in narrow, congested corridors that cannot meet travel demands with the existing number of general-purpose lanes. The choice is frequently between no improvement in corridor capacity or an HOV lane with less than desirable dimensions.

Reduced dimensions may also be present for short sections on projects that otherwise have a high level of design treatment. These may be caused by constraints that could not be removed for the HOV project. Narrow HOV lane cross sections for very short distances (e.g., under overcrossing structures with median columns) are common for many project settings.

The reduced designs are a product of a local process of investigation as to the alternatives and the impacts, both financial and physical, of various design treatments. The participating agencies determine the location and extent of design compromises. Operational improvements are frequently used as a supplement to provide high operating standards on projects with less than desirable geometric designs.

It is for these reasons that it is difficult to place the term *typical* on any HOV design element. This paper lists the desirable and reduced designs mentioned in several design guides, but designing a particular facility is often more complicated than a straightforward application of those design guidelines.

HOV Lane Configurations

The types of HOV lanes discussed in this paper are defined below and are illustrated in the accompanying figures.

Barrier-Separated HOV Lanes

Barrier-separated HOV lanes are ones that are physically separated by guardrails or concrete median barriers from adjacent mixed-flow freeway lanes (Figure 1). The opposing directions within a barrier-separated facility may also be separated by a barrier or buffer.

Buffer-Separated HOV Lanes

Buffer-separated HOV lanes are ones that are separated from adjacent mixed-flow freeway lanes with a designated buffer width of one foot or more (Figure 1). Narrow buffers of 0.3 to 1.2 m (1 to 4 ft) are either traversable or nontraversable (i.e., the buffer can be legally crossed at any point or cannot be legally crossed except at designated access points). If the buffer is sufficiently wide [3.7 to 5.3 m (12 to 15 ft)], it may be considered a refuge for disabled vehicles or for enforcement. (Neither of these uses is recommended in popular reference guidelines).

Busways

Busways are preferential roadways designed for exclusive use by buses, constructed either at, below, or above grade, and located either in a separate right-of-way or within freeway corridors (*1*) (Figure 1). Busways are not usually part of a freeway or street corridor and, thus, are not considered as part of the cross-section "integration process" that is the subject of this paper. Their design elements tend to be specific to respective bus operation needs and will not be discussed.

Contraflow HOV Lanes

Contraflow HOV lanes are ones that operate in a direction opposite that of the normal flow of traffic (commonly, the inside lane in the off-peak direction of travel) and are designated for peak direction travel during at least portions of the day (Figure 1). For freeway applications the lane is separated by plastic pylons or movable barriers.

OVERVIEW OF HOV LANE DEVELOPMENT PROCESS

Given the wide variety of types of HOV lane designs and operations that are possible, the typical developmental steps pursued for highways—planning, designing, and then operating the lanes—do not function as well for an HOV lane project.

After an HOV lane project has been determined to be feasible and some demand estimation has been performed, the concept development phase of an HOV lane project typically consists of an iterative process between operating scenarios and design treatments necessary to satisfy those scenarios. Some information on the time of operation, type of access control desired, and needs of enforcement officials is required before any design consideration can take place. Some information about the plans of transit service—both the line-haul trips and the amount, type, and location of transit or carpool support facilities—is required before the HOV lane design can be finalized.

It is the need for interaction between the operation plan and the design process that differentiates HOV lane projects from more typical general-purpose facility improvement projects. This interaction requires a broad local agency representation and base of expertise to be present in the planning discussions of HOV facility elements, but the integration of design and operations issues within the process is the key to the completion of a facility that has appropriate design standards and one that fits the needs and policies of operating and enforcement agents.

FREEWAY HOV LANE CROSS SECTIONS

There are several sources of information on the experience with HOV lane cross sections in North America. Most of this information relates to high-speed freeway-oriented HOV facilities. This section identifies current guidelines derived from operational HOV lane projects to illustrate the state of the practice regarding HOV facility cross sections.

AASHTO Guidelines

The AASHTO HOV design guidelines (*2*) were published in 1992 and represent a substantial update to the previous guide published in 1983. The previous AASHTO guidelines were developed very early in the history of HOV facilities and, as a consequence, had very few example projects from which to draw experience. The recent guide uses the knowledge gained from a wide array of operating projects to improve on the information provided to practitioners. The recent guide includes information on planning guidelines, operational considerations, and design, traffic control, and enforcement guidelines for most types of HOV lanes on freeways and arterial streets.

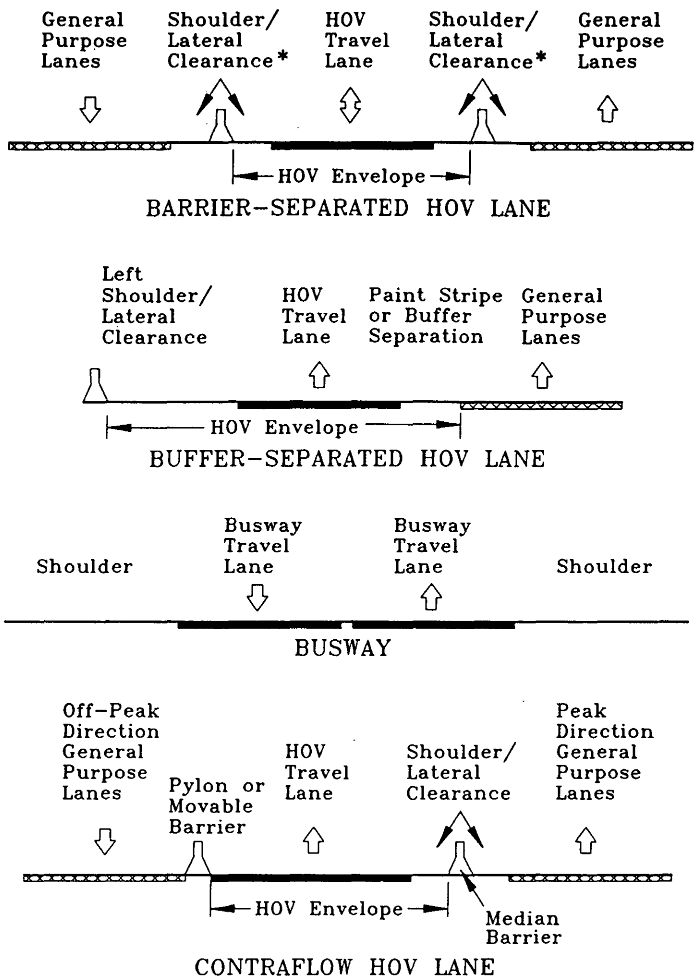


FIGURE 1 Cross-section elements of HOV lanes.

The introduction to the AASHTO guidelines indicates the usefulness of HOV facilities in providing greater movement of people in congested corridors. The introduction also indicates that the document is only a guide in planning, operating, and designing HOV lanes, not a replacement for locally applied policy and practice. The following paragraph defines the state of the practice as seen by the authors of the AASHTO guidelines.

> HOV facilities are usually incorporated into existing highway rights-of-way where width and lateral clearances may be limited. While experience has shown that some variance in design standards is possible without serious adverse effects on safety and performance, it has not been extensive enough to firmly establish new standards specifically for these types of facilities. The values presented in this guide should therefore not be regarded as absolute, but rather as the best guidance available based on experience to date (*2*).

The guide recognizes that some elements of HOV facility design are similar to those of general roadway design, and a significant portion of the design sections concentrate on the items that differ most, principally cross-section determination.

Other Sources of Design Information

The AASHTO guidelines benefited not only from the experience of operating projects but also from other studies of HOV design practices. These include the monograph prepared by Fuhs (*3*) and a technical committee report from ITE (*4*). Other studies of HOV design practice have been prepared for or by state departments of transportation and local transit agencies. Both Fuhs' monograph and the ITE informational report list cross-section and other design information for operating projects and state and local design guides. Practitioners seeking additional information on particular projects can consult those documents; this paper concentrates on the general recommendations or findings and the considerations necessary to apply the cross-section information to the implementation of an HOV lane in a travel corridor.

Operational Considerations

All the HOV lane design guides include the premise that the HOV lane design process begins with some consideration of an operating plan for the HOV lane. Four of the major issues that have a direct impact on cross-section design are addressed below. The item usually identified as the constraint to HOV lane cross-section width is the total width available for the priority facility. The manner in which the total width, or envelope, is divided between the various elements varies by type of project and local policy.

Buffer or Barrier Separation

One of the first issues to be discussed in the process is usually whether there is a need for some sort of separation between the HOV and general-purpose traffic. A reversible HOV lane in a freeway median will require a barrier, but concurrent flow treatments do not. Although barrier-separated HOV lanes typically cost more to construct and thus are not suited for interim or temporary treatments, they facilitate easier enforcement and incident management. Buffer-separated projects offer other spatial advantages, although some reflect the same envelope widths. This issue has often been decided on the basis of local preference, including the needs of freeway operations and law enforcement agencies.

One Way or Two Way

Depending on travel patterns, expected congestion, funding, and right-of-way availability, either a reversible HOV facility or a priority facility that provides benefits to both travel directions during the operating period may be appropriate. Geometric constraints and available rights-of-way also play a role in determining whether the HOV lane serves one or both directions.

Full-Time or Part-Time HOV Designation

If the HOV lane will revert to a general-purpose lane outside of the peak period, certain operational treatments such as a buffer may not be appropriate.

Carpool Occupancy Requirements

The number of people required in a carpool to be eligible to use the HOV lane, if carpools are to be a user group, has an impact on the expected volume and the treatments needed to address any problems.

HOV Lane Envelope Width

The AASHTO guidelines and the experience noted with operating projects are in substantial agreement on the desirable envelopes for HOV projects. The dimensions noted in Table 1 are consistent with those in the AASHTO guidelines (*2*), Fuhs' monograph (*3*), and the ITE informational report (*4*). For single-lane facilities the desirable width of all the treatments is between 6.7 and 8.5 m (22 and 28 ft). All the widths listed for desirable cross sections provide a continuous area to park disabled vehicles without them interfering with passing maneuvers that can be accomplished at speeds near the design speed of the facility. In the case of buffer-separated and contraflow lanes, they also provide for space to separate general-purpose and HOV traffic. Wider dimensions of up to 8.5 m (28 ft) have been used for special local conditions such as snow storage, enforcement, or space for future implementation of a very narrow two-lane HOV configuration.

Most design guides include some information regarding reduced cross sections. As previously noted, this dimension is not standard and is the product of local discussion and agreement on the definition of reduced cross sections and the situations in which such cross sections are applicable. The reduced dimensions included in Table 1 are representative of those agreements and the operating HOV lane projects in North America. The dimensions for the barrier-separated projects provide an area to park a disabled vehicle and allow for passing; this is a requirement for any section of significant length on a facility that is completely enclosed by barriers. The narrow buffer project width allows for separation from the median barrier and from the general-purpose lanes. The contraflow project dimension is in operation on two HOV projects in the New York City area and is planned for one project planned in the Boston area; other contraflow projects in California and Texas have been able to include a shoulder for broken down vehicles as part of the basic cross section.

Barrier-Separated, Single-Lane, Reversible HOV Facility

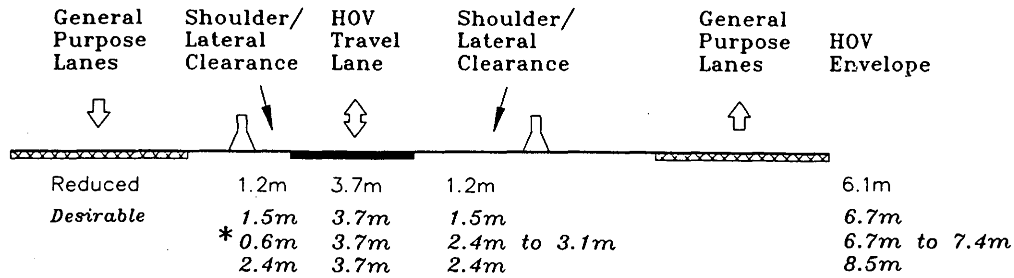
Figure 2 shows how the envelope is typically striped for single-lane HOV facilities that are reversible. Equal distances on each side of the travel lane have been used to provide maximum separation from the barrier when insufficient width for a shoulder exists. The reduced envelope of 6.1 m (20 ft) is such a case. Disabled vehicles would be parked against one barrier and HOV lane traffic would pass on the other side, crossing over the shoulder stripe in the process. This operating plan has not posed any problems in the Houston HOV lane system, where this cross section has operated since 1984.

A 6.7-m (22-ft) wide envelope appears to be a dividing line for the shoulder striping decision—it may be striped for equal lateral clearances of 1.5 m (5 ft) or with a 2.4-m (8-ft) parking shoulder on one side. With cross sections of greater than 6.7 m (22 ft), a full-width parking shoulder would be provided. The provision of one wide area has the advantage of alerting motorists to the probable location of parked vehicles. The 8.5-m (28-ft) HOV lane

TABLE 1 Summary of HOV Lane Envelope Widths

	Width of HOV Lane Envelope	
	Desirable	Reduced
Barrier-Separated, One-Lane	6.7 to 8.5 m	6.1 m
Barrier-Separated, Two Lane	13.4 m	11.0 m
Buffer-Separated	7.9 m	4.9 m
Contraflow	6.7 to 7.3 m	3.7 m

Note: 1 meter = 3.28 feet



1 Meter = 3.28 Feet

*Note: Parking shoulder could be placed on either side of this cross section

FIGURE 2 Single-lane, reversible, barrier-separated HOV lane cross-section dimensions.

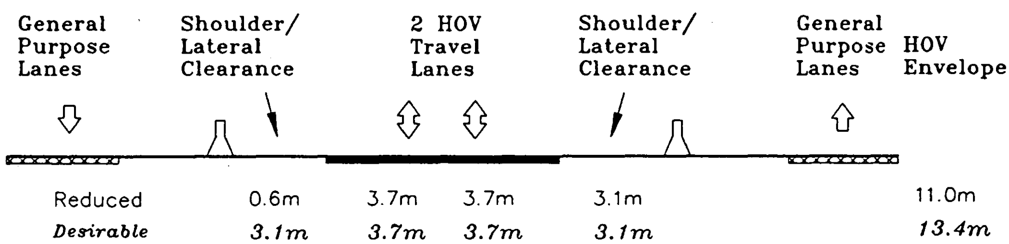
cross section would be striped, with a 2.4-m (8-ft) shoulder on each side.

Barrier-Separated, Two-Lane, Reversible HOV Facility

The higher traffic volumes on two-lane HOV facilities suggest the need for at least one full shoulder in even a reduced cross section. The higher volume increases the likelihood of an incident and magnifies the consequences should that incident block a travel lane. The dimensions in Figure 3 show a 0.6-m (2-ft) lateral clearance in the reduced cross section and a second full shoulder in the desirable cross section. Various widths of between 11 and 13.4 m (36 and 44 ft) are possible, with the important note that shoulder widths of between 1.2 and 2.4 m (4 and 8 ft) should be avoided if possible. Motorists might be tempted to park in such a space, even though it is not sufficient for a safe emergency shoulder.

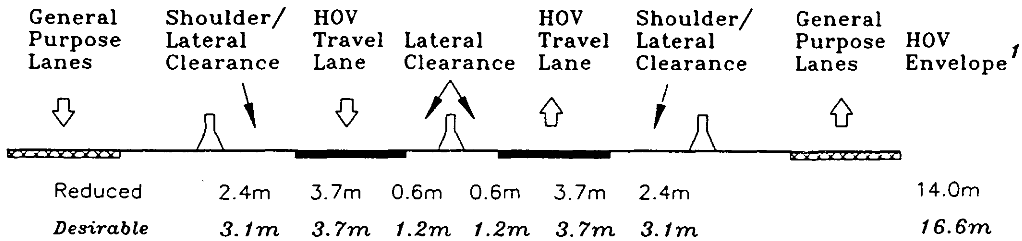
Barrier-Separated, Two-Way HOV Facility

Each lane of a two-way HOV facility operates in the same direction all the time, unlike a reversible lane. This leads to the provision of a wider area to the right side of the HOV lane in all design guides. The 6.7-m (22-ft) envelope width in the reduced cross section (Figure 4) is 0.6 m (2 ft) wider than the reduced reversible cross section. Although no project has implemented such a cross section, it could be argued that a 6.1-m (20-ft) envelope would be appropriate for a reduced cross section even when that resulted in a 1.8-m (6-ft) right shoulder and a 0.6-m (2-ft) left lateral clearance. Such a configuration would satisfy the



1 Meter = 3.28 Feet

FIGURE 3 Two-lane, reversible, barrier-separated HOV lane cross-section dimensions.



1 Meter = 3.28 Feet

[1] Note: Includes 0.6m for median barrier

FIGURE 4 Two-way, barrier-separated HOV lane cross-section dimensions.

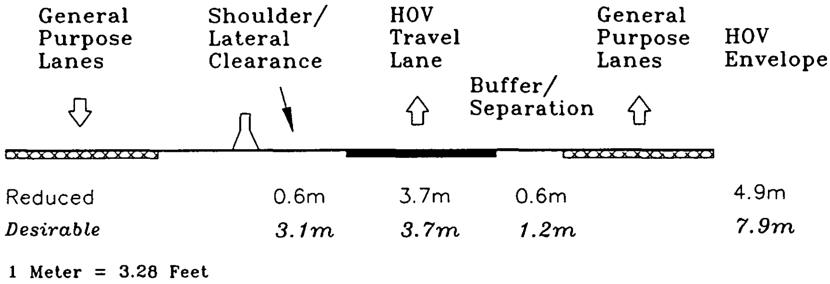


FIGURE 5 Buffer-separated HOV lane cross-section dimensions.

need for motorists to be able to pass a disabled vehicle on a 6.1-m (20-ft)-wide HOV lane and would have an area where motorists could expect disabled vehicles to be parked. The advisory against designing shoulders of between 1.2 and 2.4 m (4 and 8 ft) would be overridden by the tendency of drivers to move away from parked vehicles in order to pass them, regardless of the presence of a paint stripe delineating a lateral clearance.

The 8.0-m (26-ft) envelope in the desirable cross section is also wider than the envelope in the reversible cross section. The small additional cost of providing a 3.1-m (10-ft) shoulder and a 1.2-m (4-ft) lateral clearance on a facility that will be approximately 15 m (50 ft) wide (for both directions) seems to suggest that such an improvement from the reversible guidelines is reasonable. The AASHTO guidelines include only a 0.6-m (2-ft) lateral clearance with a 3.7-m (12-ft) shoulder, resulting in the same envelope width.

Buffer-Separated HOV Facilities

A greater portion of buffer-separated HOV facilities than barrier-separated lanes are developed as retrofit treatments, and the cross-section configurations are consequently more varied.

Many HOV lanes operate with less than the reduced cross section shown in Figure 5. For lanes with full-time HOV designation the provision of some buffer width appears desirable. The placement of the HOV travel lane between a lateral clearance and a buffer of equal width is similar to that of the narrow reversible barrier-separated HOV lane.

The desirable cross section includes a full shoulder on which to park disabled vehicles on the median side of the HOV lane. The wide parking area for disabled vehicles is consistent with the barrier-separated HOV lane standards. A 1.2-m (4-ft) buffer is the maximum that can be provided if parking between the HOV and the general-purpose lanes is to be discouraged. A 1.2-m (4-ft) separation can provide motorists in HOV lanes with a warning that a general-purpose motorist is about to enter the HOV envelope and allow the driver in the HOV lane to take avoiding action.

Nonseparated HOV Facility

One type of HOV lane that cannot conform to the idea that the provision of a buffer is a positive aspect of an HOV lane is the HOV facility that is designated for use as both an HOV and a general-purpose travel lane. If a buffer were provided in this situation, there could be confusion over the role of the buffer area during general-purpose operation, another illustration of the need for operating information before HOV facility design begins.

The cross section in Figure 6 shows situations similar to those in the buffer-separated projects: a narrow envelope in the reduced cross section and a full shoulder in the desirable cross section. As with the buffer-separated projects, there are in operation lanes with less than a 1.2-m (4-ft) left lateral clearance.

Cross-Section Compromises

The retrofit nature of many HOV projects has influenced many design elements, but it has also influenced the nature of the design guidelines used in the development of the projects. If there is a recognition of the inevitability of constraints on the HOV lane cross section, a set of trade-offs that could be used to guide the design team could be agreed upon. The Fuhs monograph (*3*) pioneered this concept, which was also included in the ITE guidelines (*4*) and other state and local guidelines. The AASHTO guidelines (*2*) do not reflect such a concept.

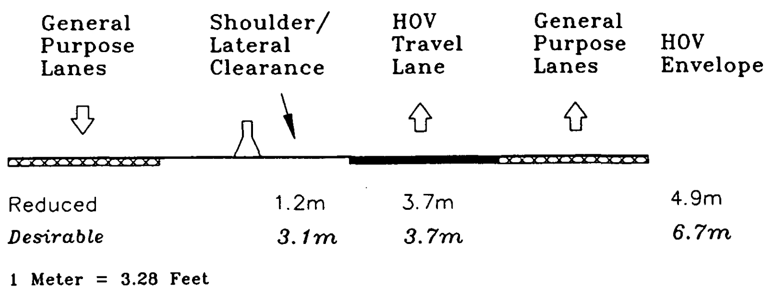


FIGURE 6 Nonseparated HOV lane cross-section dimensions.

Table 2 gives an example of how the compromise technique could aid HOV facility designers in the consistent application of design guidelines to a variety of situations that could require less than desirable dimensions. The list in Table 2 reflects both HOV and general-purpose cross-section dimensions because that is the usual pattern in compromises—the most important elements are retained regardless of their designated use, and the idea of "getting the most bang for the buck" is applied to the entire cross section. Each project may have a different set of compromises, but if they can be agreed upon in advance rather than requiring case-by-case review, the design process can proceed with less delay and more consistency.

The example in Table 2 is derived from the Fuhs (*3*) and ITE (*4*) publications and indicates the general order of compromise in the operation or design that appears in the projects. If the order is reversed, a general priority of cross-section elements can be derived. After the initial HOV envelope, the provision for one freeway shoulder, wider freeway lanes, a wider HOV envelope, and a second shoulder are listed.

ARTERIAL STREET HOV CROSS SECTIONS

Design information on arterial street HOV projects is not as readily available or as plentiful as that on freeway HOV projects. The AASHTO guidelines provide some information as to the types and configurations of the HOV lanes that may be present, but most arterial street HOV projects are retrofit projects that have cross-section elements similar to those of general-purpose arterial streets. The Fuhs (*3*) and ITE (*4*) publications and most other local and state standards do not address arterial street HOV lane designs.

A major part of this uncertainty is the lack of bus and carpool HOV lane treatments on arterial streets. There are many cities with downtown streets or lanes designated for bus-only use during all or part of the day. There are also some arterial street bus-only treatments over significant distances (i.e., more than a few blocks), but on very few arterial street priority lanes are carpools allowed as users. Any guidelines that have been developed are therefore generally based on bus projects and standards that are applicable to general-purpose-street cross sections as well as HOV treatments.

This section focuses on a description of the general types of arterial street priority treatments and a discussion of the issues involved in developing cross-section designs for HOV lanes on arterial streets. The experience does not exist to be able to develop recommendations on the desirable cross sections for these treatments as they are applied to HOV projects. It should also be recognized that much of the success of these lanes will depend on operational enhancements that are implemented regardless of the cross-sectional width of the facility.

Types of HOV Facilities

There are three general types of HOV facilities on arterial streets—median or center lanes, concurrent flow lanes, and contraflow lanes. As with freeways, there are also roadways for the exclusive use of HOVs that are separate from streets. These are usually reserved for buses rather than buses and carpools.

Median or Center HOV Lanes

Median or center HOV lanes are usually reversible and can be separated by curbs or barriers or can be unseparated from general-purpose traffic. They are implemented when there is a directional imbalance in travel volume and congestion. A significant problem for these type of facilities is the loading and unloading of passengers on buses in the center lane.

Concurrent Flow Lanes

Concurrent flow lanes, which operate with the flow of adjacent traffic, are usually next to the curb or median, depending on the specific circumstances and objectives of the project. The principal determinant may be the type of bus service that will be favored; local service benefits from curb lanes and express service will usually work better in median lanes. The lanes are not separated from traffic in the same direction and may not be separated from street traffic in the opposite direction.

Contraflow Lanes

Contraflow lanes, which operate against the flow of adjacent traffic, can also operate next to the curb or median. Some positive separation (curbs, plastic posts, or barriers) is more desirable for contraflow lanes than for concurrent flow lanes, especially for facilities that will have a high carpool volume. Contraflow lanes may be implemented on either two-way or one-way streets.

Arterial Street HOV Lane Design Issues

With the lack of operating experience for HOV lanes on arterial streets, there is also little experience on the types of design treat-

TABLE 2 Example HOV Lane Cross-Section Compromise

Order of Compromise	Element
First	Reduce freeway left shoulder to 0.6 m
Second	Reduce freeway right shoulder to 2.4 m
Third	Reduce HOV lane envelope to 6.1 m
Fourth	Reduce freeway lane widths to 3.4 m
Fifth	Reduce freeway right shoulder to 0.6 m
Sixth	Reduce HOV lane envelope to 3.7 m

Note: 1 meter = 3.28 feet

ments that are appropriate for different types of situations. The guidance provided by the AASHTO guidelines (*2*) will be incorporated into this discussion of the significant issues associated with the design of HOV lanes on arterial streets. As with freeway HOV facilities, arterial street HOV lane design requires some extensive knowledge of the operation plan and the objectives of all participating agencies.

HOV Lane Width

Arterial street HOV lanes are usually retrofit treatments and therefore are subject to the constraints of the existing street geometries. Priority lanes that are 3.7 m (12 ft) wide are desirable in most applications (*2*), but when heavy pedestrian flows are adjacent to the HOV lane, the HOV lane may be 4 or 4.3 m (13 or 14 ft) wide. The AASHTO guidelines (*2*) list 3.4-m (11-ft) HOV lanes as being acceptable in restricted locations, which could be frequent in arterial situations. In some applications there may be a need for two HOV lanes, but more often the need will be for bus turnouts to allow carpools and express buses to bypass local buses at stops.

Separation of HOV and General-Purpose Traffic

Median and contraflow HOV lanes may require some positive separation between the two types of traffic. Curbs, concrete barriers, or plastic posts in the pavement can be used to delineate the HOV lane. These separators can be used to reduce conflicts and violations of the vehicle occupancy restrictions. Plastic posts or movable concrete barriers would be used if the HOV lane were not in operation for the full day. With frequent intersections, there should not be a need for width to pass disabled vehicles, but separators may not be desirable if they would result in lane widths that are less than desirable.

General-Purpose Traffic Use

With many central business district bus lanes there is a provision for the use of the priority lane by general-purpose traffic for short sections as turn lanes. Off-peak use may be permitted by general traffic. For arterial street HOV lanes, similar uses may be permitted to improve traffic flow on both priority and general-purpose lanes. If turns across curb HOV lanes are not permitted, significant redirection of traffic may result. If the general traffic cannot turn from the HOV lane and turns are permitted, the traffic streams would cross, which would not be desirable. If the arterial HOV lane is not designated for use during peak periods only, some provision for the loading of goods should be made. It is desirable that at least two general-purpose lanes remain in the HOV direction of travel.

Access and Egress

Entry and exit from the HOV lane can be provided at the cross-street intersections on arterial streets. Upstream treatments at initial access points should require entering vehicles to make a definite movement to access the HOV lane to reduce inadvertent use of the lane. Downstream treatments should allow the vehicles in HOV lanes to move into an unoccupied lane rather than merge into the general-purpose traffic stream. Access and egress along the lane can be at any point, which increases the possibility of violation and the importance of enforcement.

Signalization and Signing

Most of the designation of the priority lane will be by signing and marking. Vehicular traffic must be alerted to the priority designation for any type of facility. Overhead and curbside signing and pavement markings must be frequent and visible. Signing to alert pedestrians to the priority lane is especially important in curbside contraflow lanes, where buses and carpools approach from the direction opposite that expected by the pedestrians.

The use of priority signalization plans for HOV treatment may not be possible for congested corridors, but a scheme that recognizes the increased movement of people represented by the HOV lane will improve vehicular flow on the arterial street. Signalization at cross streets will be simplified if separate turn phases are not required for HOVs. This will typically require that general-purpose vehicles be allowed into or through the HOV lane near intersections. High-volume intersections may require additional lanes to handle HOV and turning traffic.

Bus Stop Location and Passenger Loading

Curbside HOV lanes can use normal bus stop designs, although some provision for bus stop turnouts may be required. Median lanes require some significant provision for bus loading and unloading if that is to be part of the operating plan. Median lanes, however, will probably be developed for express lanes, and bus stops will not be a significant part of the design. The platform width should be at least 1.5 m (5 ft), but if traffic flows are on both sides of the platform, 3.1 m (10 ft) is recommended (*2*).

Enforcement

Frequent access possibilities and turning points allow HOV lane violators easy entry and escape from the HOV lane, which makes enforcement difficult and important. Enforcement agencies should be included in the design team, and alternative means of enforcement such as ticket-by-mail or identification of violators by motorists (to get a notice in the mail rather than a citation) should be considered. The constrained width in arterial treatments will probably mean that it may be difficult to implement separate areas for enforcement, but when possible they may improve the appearance and efficiency of the priority lane.

RECOMMENDATIONS FOR FUTURE RESEARCH

Several aspects of HOV lane design mentioned in this paper refer to additional information that is needed to improve the state of the practice. This section discusses the major aspects for which research will provide improvement to the design of future HOV projects.

Arterial Street HOV Lane Design

The several aspects of arterial street HOV lane design discussed in this paper will be more fully developed as more of these types of treatments are implemented. Freeway HOV treatments have benefited from several different applications and a variety of different design standards; arterial street HOV treatments will also benefit from such a range of treatments.

Safety and Cross-Section Compromise Studies

Studies of the impact of narrow cross sections and the best tradeoffs that should be made in situations of constrained width will benefit future project designers. More information on the experiences from the different types of HOV treatments will be required as HOV lanes become more widely utilized.

Enforcement Provision

Although the design of enforcement locations is often determined locally, more information on the types of treatments that have been successful is important. The interaction of design elements and enforcement effectiveness should be a prominent part of any design document, with the emphasis being on the early involvement of the enforcement agencies in the planning and design of the project.

Buffer Width

Designers appear to be nearing a consensus that a 1.2-m (4-ft) buffer is the best dimension for freeway projects that are designated for full-time HOV use. Experiences of previous projects have led California Department of Transportation officials to choose this dimension for many of their recent projects that operate on a 24-hr basis. Research may confirm the desirability of this cross section and identify its applicability to arterial street HOV projects.

Access and Egress Design

The interaction of cross section and access and egress design is an important element in successful HOV projects. Balancing the exclusivity of the HOV project, enforcement provisions at entry points and along the lane, and the cost-effectiveness of various types of access treatments has been a significant part of the HOV design process. The access and egress treatment design is part of the interactive process between planning, operation, and design that characterizes HOV project development. More guidance on the amount of the total project cost that should be devoted to access and egress would assist designers.

Signing and Marking

There are very few standardized signs or markings in practice on HOV lanes. This often causes HOV lane designers to provide additional widths or other exceptional treatments to deal with motorists unfamiliar with HOV lanes. Standardized signing and marking may reduce the redundancy required in some types of HOV designs, particularly arterial street designs.

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Factors Affecting Selection of Lane Width and Shoulder Width on Urban Freeways

THOMAS URBANIK II

Over the years a set of desirable design standards has evolved for lane width and shoulder width on freeways. These standards have been applied extensively, and a level of comfort has developed in which the use of full design standards results in roadways that are safe and operate satisfactorily. The use of these full standards is preferred and, all things equal, should form the basis for roadway design. However, especially in the upgrade and reconstruction of existing roadways—many of which were not originally built to full standards—a variety of competing factors begin to require attention. Some of the issues that justify consideration are (a) traffic operations, (b) traffic safety, (c) maintenance, (d) enforcement, (e) project cost, (f) public acceptance of the project, and (g) environmental issues. In effect a complicated trade-off analysis begins to take place. Although it is clearly desirable to construct a roadway that will be safe to operate and maintain, frequently other issues such as cost and environmental impacts can be greatly mitigated if something less than full design standards are used, at least at selected locations on a project. The effects of these various factors on the selection of lane width and shoulder width on urban freeways are assessed.

Over the years, a set of desirable design standards has evolved (*1*). These standards have been applied extensively, and a level of comfort has developed in which the use of full design standards results in roadways that are safe and operate satisfactorily. The use of these full standards is preferred and, all things equal, should form the basis for roadway design. This is particularly true when a proposed roadway improvement represents an "ultimate" plan for that particular facility.

However, especially in the upgrade and reconstruction of existing roadways—many of which were not originally built to full standards—a variety of competing factors begin to require attention. Some of the issues that justify consideration are (a) traffic operations, (b) traffic safety, (c) maintenance, (d) enforcement, (e) project cost, (f) public acceptance of the project, and (g) environmental issues. In effect a complicated trade-off analysis begins to take place. Although it is clearly desirable to construct a roadway that will be safe to operate and maintain, frequently other issues such as cost and environmental impacts can be greatly mitigated if something less than full design standards are used, at least at selected locations on a project. In fact, in addressing project feasibility, the choice may well be that, by using reduced standards, a meaningful project improvement, although perhaps not the perfect improvement, can be implemented. If strict adherence to all standards is mandated, it may be that no project improvement is feasible. Consideration needs to be given to the marginal benefits gained from marginal increases in expenditures. Nevertheless, in considering the use of reduced standards, it is important to be comfortable that, by so doing, traffic operations and safety will be acceptable.

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Although only limited data exist concerning the use of reduced standards, the experience with reduced lane and shoulder widths has generally been good. Despite the record, there is continuing concern as to the appropriateness of reduced standards given the relatively small amount of relevant evaluation that has been performed. The NCHRP has under way a 27-month, $300,000 study (Project 3-43) to help provide additional understanding of these complex issues.

Thus as a general approach it is suggested that full standards be used initially as the basis for highway design. On the basis of the conditions that occur at specific locations along a project, it may well be appropriate, with justification, to deviate from full standards at those locations to permit the project to move forward cost-effectively—if experience elsewhere leads to a reasonable assurance that the use of reduced standards will still result in a roadway project that is safe and operates acceptably. For this approach to work it requires that those in decision-making positions be receptive to the use of reduced standards when appropriate. Technical staff need to provide the necessary supporting justification. At least to an extent this is a statement of the obvious. Given the limited funds relative to the needs it has been suggested (*2*) that full compliance with AASHTO standards (*1*) is not always the most effective use of available space. One means of providing additional capacity quickly and inexpensively has been to reduce or eliminate shoulders and to narrow lanes. Even though the safety records of those projects that have been evaluated are generally good, there is continuing concern as to the appropriateness of shoulder removals given the limited amount of evaluation.

Many older multilane urban freeways were built without full left shoulders, right shoulders, or both, and continue to operate that way today. Examples include the Triboro Bridge approach in New York City, Lake Shore Drive in Chicago, Bayshore Freeway in San Francisco, and I-95 across the Bridgeport Harbor in Connecticut.

Most newer freeways provide full left and right shoulders when there are six or more lanes. The provision of these shoulders is consistent with AASHTO design guidelines and Interstate highway system criteria. However in the 1960s mounting congestion on freeways in the Los Angeles area caused the California Department of Transportation, known as Caltrans (then the Division of Highways), to look at ways of increasing freeway capacity within existing paved rights-of-way. This led to the provision of additional travel lanes by narrowing lanes to 11 ft in a few cases, 10.5 ft, and reducing shoulder widths. By 1989 there were 180 miles (one-way direction) or urban freeways with nonstandard shoulders and narrow lanes in Los Angeles (*3*).

Agencies in other urban areas and states followed California's initiative. Houston subsequently provided additional travel lanes by narrowing shoulders. By 1978 increased capacity through the use of shoulders and narrow lanes had been implemented in Denver, Nashville, Pensacola, Boston, New York City, Providence (Rhode Island), and Portland (Oregon) as well as the Hartford area in Connecticut (*4–12*). Since then projects have been implemented in Chicago, Dallas, and Phoenix (*7*).

At most of the projects the anticipated traffic engineering benefits were realized. For example, the added lane increased the capacity of the section with a resultant decrease in total travel times, an improvement in the level of service and a reduced number of traffic conflicts. Some projects were designed to provide space for high-occupancy-vehicle (HOV) operations, whereas others were modified to provide emergency parking lanes.

ISSUES

Although safety is often the reason offered in opposition to the use of reduced standards, several issues are interrelated to safety and operations. In addition to safety, issues include lane positioning, frequency of stops, maintenance, enforcement, capacity, weaving, and sight distance. The following discussion will review the available information on these issues.

Safety

The accident experience initially reported by McCasland and Biggs (*6*) in 1980 and updated by Urbanik and Bonilla in 1987 (*7*) for 24 projects indicated that most of the sites experienced decreased accident rates after the projects were implemented. Houston and Los Angeles are prominent, with the largest number of documented cases; however, documentation varies substantially from one project to another.

McCasland and Biggs (*6*) noted that narrowing of the lanes to 11 feet (or occasionally 10.5 feet) while maintaining shoulders did not change accident rates. Projects in which one or both shoulders were eliminated during peak periods did not experience increases in accident severity, although it is important to note that unpaved emergency parking space existed beyond the right lane except at bridges. There were questions however about future effects and whether increasing volumes would bring back the level of congestion that existed before the improvements. Since the project to increase capacity also brought about an immediate improvement in the level of service on the freeway, it was believed that the congestion reduction benefits overshadowed the negative effects of reducing or eliminating shoulders.

A review of accident rates on the California projects (*5–7*) revealed that higher accident rates had not materialized several years after lanes were narrowed and left shoulders were removed. The projects represent long-term operational improvements because operations within the sections never returned to stop-and-go operation, even though the total volumes eventually approached or exceeded those before the improvements. The reason for the permanent nature of the improvement can be attributed to the metering effect of the upstream interchanges. The result is that the per lane volumes in the improved section remained at levels below the preproject level.

An important issue pointed out first by Urbanik and Bonilla (*7*) and later by Levine et al. (*13*) is the issue of accident migration. Urbanik and Bonilla demonstrated that accident migration is not a problem on well-designed projects. The conversion to reduced left shoulder widths and 11-ft lanes should be based on an analysis demonstrating that operational problems are not being relocated to another point in the highway system; traffic engineering studies are needed to ensure that proposed improvements do not create new problems.

Lane Positioning

The elimination of shoulders potentially has an impact on operations in a number of ways, including the lane positioning of vehicles. Urbanik and Bonilla (*7*) conducted a study on the Katy Freeway (I-10) in Houston to evaluate the impact of a concrete median barrier on lane placement. A comparison of vehicle placement on the lane relative to the left edge line was made on a section with a full shoulder and one with a 1-ft shoulder. The findings from the study indicated that driver performance relative to shoulder width could be measured and used as an indicator of minimum desirable shoulder widths. The initial study showed a "shy" distance of about 1 ft with a 1-ft shoulder. They suggested that the prevailing practice of desiring a 2-ft minimum shoulder width appears correct.

Urbanik (unpublished research, Texas Transportation Institute, College Station, August 1989) later evaluated vehicle placement within the left lane of a freeway at three locations along the southbound I-45 (Gulf Freeway) in Houston as it approaches I-610 South (South Loop Freeway) using means more sophisticated than those used in the initial study. Urbanik concluded that lane placement is affected by lane width, distance to barrier, darkness, and the volume in the adjacent lane. However, the data base was insufficient to make specific recommendations other than the apparent desirability of providing greater shoulder width than the existing width of less than 1 ft.

Characteristics of Stopped Vehicles on Urban Freeways

The existence of a shoulder provides an opportunity for vehicles to stop outside a traffic lane. These stops may be voluntary or involuntary; this distinction is important because voluntary stops may be deferred.

Vehicle Stop Rates

Hauer and Lovell (*3*) conducted a study on safety measures aimed at reducing accidents occasioned by vehicles stopped on freeway shoulders. They concluded that for every emergency (involuntary) stop by a passenger car there are seven to eight leisure (voluntary) stops, and for every emergency stop by a truck there are about five leisure stops. Trucks stop for emergencies almost three times more frequently than cars. It was cautioned that these data primarily represent daytime stops. Detailed data collected in Houston (*14*) suggested that one vehicle breakdown can be expected to occur about every 35,000 vehicle mi of travel.

Surveys of Drivers Stopped on Shoulders

Urbanik and Bonilla (*7*) conducted two data collection efforts. In both studies sections of roadways were periodically observed for stopped vehicles and data were collected on the type of vehicle and purpose and length of the stop. In the first study main-lane stops appeared to be disproportionately represented in the section without any shoulders. Main-lane stops were observed at a rate of 1 in 167,196 vehicle mi overall, in comparison with a rate of 1 in 16,129 vehicle mi for the one section without any shoulders. In the second study they found that use of the left shoulder is infrequent even on sections with fully paved inside shoulders. The one observed main-lane stop was on a section with no left shoulder.

Urbanik and Bonilla (*7*) used a floating-car observer to survey stopped vehicles by either handing a questionnaire to the driver or placing the questionnaire on the windshield of the vehicle stopped on the left shoulder. The most significant finding was the higher rate of involuntary stops for those using the left shoulder. This result is consistent with the belief that drivers prefer to use the right shoulder when the option exists.

Data from Caltrans (*15*) for the Los Angeles Hollywood Freeway (State Route 101) indicate a rate of 1 stop per 9,800 vehicle mi on the basis of 337 observed stops. The disablement rate (stops longer than 8 min) was 1 per 25,000 vehicle mi. The data were collected by stationary observers.

Again, Texas data (*14*) suggest a disablement rate of about 1 per 35,000 vehicle mi of travel. This data base is extremely good since it measures only vehicle breakdowns and is based on extensive data collection. Of the breakdowns 35 percent were due to a flat tire, 18 percent ran out of gas, 15 percent had overheated engines, 12 percent had electrical or mechanical problems, and 20 percent broke down for other reasons.

Maintenance and Enforcement Issues

The reduction of shoulder width reduces the functional usefulness of shoulders for police enforcement operations, highway department operations and maintenance, incidence responses, and other emergency service activities. In 1986 the California Highway Patrol (CHP) and Caltrans (*16*) conducted surveys of

- CHP officers and supervisors responsible for patrolling urban freeway segments,
- Caltrans operations and maintenance supervisors responsible for the particular freeway segments,
- Relevant incident response team personnel,
- Fire suppression and rescue service teams, and
- Towing service operators.

A total of 122 questionnaires from field management and supervisory personnel formed the basis of the report. The findings indicate that provision of at least one shoulder is important.

CHP personnel indicated that the efficiency and effectiveness of enforcement operations are substantially affected when shoulders are not available. They also indicated that the safety of individuals involved in an enforcement-related activity is a major concern. Some officers are reluctant to initiate an enforcement action in areas without shoulders. Almost all the officers reported that they try to make motorists drive to a safe location, but frequently the violator does not understand or want to comply with the officer's direction. CHP recommended standard shoulder widths of 10 feet in open areas and 12 ft in areas bordered by barriers. CHP recommended that, when shoulder reduction must occur, only median shoulders should be removed. When no shoulders can be provided, spacious turnouts should be provided.

Maintenance operations, which are commonly accomplished from a shoulder, usually require closing a freeway lane when there is no shoulder of adequate width to accommodate personnel and equipment. Closing a freeway lane can result in more congestion. More personnel, equipment, and time are required to provide motorists with advance warning. Although a highly accurate cost comparison between maintenance activities performed at locations with and without shoulders was not available, an increase of 50 to 250 percent in the cost was estimated by Caltrans. The increase was based on the following: (a) increase in personnel use, (b) increase in equipment use, (c) decrease in average daily production, (d) increase in labor cost when Saturday or Sunday work results in overtime pay, and (e) increase in the number and extent of damage to highway facilities. Eighty-five percent of the respondents indicated that elimination of the left shoulder, if a shoulder must be eliminated, would create fewer maintenance problems. This is particularly true when a median HOV facility can be used for maintenance during nonpeak periods. Forty-two percent of the maintenance personnel considered 10 feet to be the minimum shoulder width for maintenance activities, whereas 28 percent considered 8 feet as the minimum shoulder width for maintenance activities.

The report included several suggestions and recommendations on methods that could be used to mitigate problems for maintenance personnel when there is no freeway shoulder. Sample recommendations include (a) more input from maintenance personnel, (b) replace cable and metal median barrier with concrete barrier, (c) eliminate or replace high-maintenance glare screens on concrete barriers, (d) construct turnouts, (e) provide adequate structural strength for shoulders that are used as lanes, (f) relocate drainage inlets, and (g) plant low-maintenance landscaping.

Freeway shoulders are used by maintenance as well as other emergency equipment to reduce response time to the scene of incidents when lanes are congested or blocked by traffic. Because of the increase in response time when shoulders are reduced in width, motorists are exposed to hazardous conditions, traffic congestion, and delays for longer periods of time than if there were shoulders.

Tow-truck operators inside the city of Los Angeles were surveyed to determine the impact of shoulder availability on tow-truck operations. An average of 1,328 service calls per month were reported by operators for 9 of 18 police divisions in the city of Los Angeles. Although it does not directly relate to the issue of narrow shoulders, the extent of calls does indicate the need to provide at least one full shoulder.

Fire suppression and rescue personnel have indicated that urban freeway shoulders reduce their response times during periods of heavy traffic or when traffic is congested because of an incident. They responded to 1,930 freeway emergency situations during the 12-month period ending June 30, 1987. The shortest response times are on freeways with both left and right shoulders. The next shortest response times are on freeways with only a right shoulder; this is followed by the response times on freeways with only a median shoulder. Average response times on freeways with no shoulders are more than double those on freeways with both shoulders. Again the need to maintain at least one shoulder is important.

Urbanik and Bonilla (*6*) also demonstrated that, on the basis of typical vehicle breakdown rates, sections with no shoulders were likely to cause as much delay because of incidents as they would see because of added capacity. Possible exceptions to this basic principle would be short sections (e.g., underpasses) and facilities with special mitigation, such as that sometimes provided at major bridges and tunnels.

Capacity and Weaving

The 1985 *Highway Capacity Manual* (*17*) provides reduction factors for computing capacity because of narrow lanes and narrow shoulders. A footnote to Table 3.2 of the manual indicates that adjustment factors for lateral clearance may not be appropriate with high types of barriers. Judgment is suggested in applying the factors. Newman (*18*) in 1985 concluded that 11-ft lanes have no effect on the level of service and safety and no measurable impact on capacity.

Urbanik et al. (*19*) presented freeway capacity data that included 12 high-volume flow rates observed in Texas. Only 2 of the 12 observations were greater than the 2,349 vehicles per hour per lane that were observed on a section with 11-ft lanes and full shoulders. It should also be noted that the high-volume section with 11-ft lanes is located just downstream from a weaving section with 11-ft lanes and no left shoulder. There is no indication of weaving problems associated with the high flow rates.

Weaving analysis in general has been the subject of numerous studies. The most recent and promising work was done by Cassidy et al. (*20*) in California. They concluded that existing methods do not have a strong predictive ability. The research of Cassidy et al. suggests that weaving capacity is a function of the number of lanes (i.e., no point on the freeway should have more than 2,200 passenger cars per hour per lane), geometric configuration (e.g., the use of optional lanes at multilane exits), and weaving section length. An 11-ft lane width is unlikely to have any more of an impact on capacity in weaving sections than it has on the capacity of basic freeway sections. However, weaving is more a question of the number of vehicles per unit length (density) than it is a question of lane positioning. That is to say, weaving problems occur when too many vehicles try to use the same lane at the same time or when a vehicle slows to look for a gap in an adjacent lane.

Sight Distance

The issue of horizontal sight distance is based on the basic idea of providing adequate stopping sight distance. Virtually no research has been done on the issue. Leisch (*21*) has made recommendations concerning horizontal sight distance that are based on making the most effective use of the available space. Because of the directionality of sight distance obstructions, the use of shifted alignments can sometimes reduce the sight obstruction.

A critical evaluation of the horizontal stopping sight distance model suggests that it may be conservative relative to typical freeway applications. The likelihood that a 6-in.-high object entering heavy traffic is not immediately hit, regardless of stopping sight distance, seems remote. Under light traffic conditions on a multilane freeway, lane changing is as reasonable an action as any.

The more likely hazard on a busy freeway is a vehicle stopped because of a breakdown or congestion. Congestion is a safety problem that can be amplified by inadequate sight distance (horizontal or vertical). One could argue that maintaining stopping sight distance for stopped vehicles is a reasonable compromise when reconstructing an existing facility for which improving the horizontal alignment is not a practical alternative.

However, the issue of horizontal sight distance is complicated by other factors. The sight obstruction is typically a concrete median barrier or a bridge column. If the obstruction in the curve is a bridge column, it is likely that object height is irrelevant. With a bridge pier there is no advantage to a higher object for horizontal sight distance. With a median barrier object height is important because it may be possible to see over the barrier. However, seeing over the barrier is complicated. Superelevation and vertical curves further complicate sight lines.

AASHTO suggests that taillights 1.5 to 2.0 ft high, and a 1.5-ft taillight height is appropriate as a design value. Given a 32-in. concrete median barrier, visibility is marginal at best under ideal conditions for a 2.0-ft taillight height and would likely be obscured for a 1.5-ft object height. Some barriers may be higher because of differential elevations between roadways or glare screens. The high-mount brake light is of some help, but only when the brake lights are on. It would seem to be necessary to have roadway lighting to argue for an object height of more than 1.5 ft.

Research by Urbanik et al. (*22*) on two-lane highways indicated that the presence of vertical sight distance restrictions did not, by itself, cause a safety problem. The AASHTO stopping sight distance model alone is not a good indicator of accident rates on two-lane highways. Higher accident rates were found to exist at access points such as driveways and intersections. Extrapolating the findings for two-lane roadways to freeways would suggest that reduced sight distance would likely be a significant consideration only when stopped vehicles have a high probability of being present.

In summary being able to provide sight distance by seeing over a median barrier is a complex situation. Good lighting, the absence of columns or high barriers or glare screens, and no elevation distortions such as vertical curves or superelevation appear to be prerequisites for good sight distance. Sections exist in both Texas and California where horizontal sight distance is not adequate in sections with reduced design standards. Those sections have not been explicitly evaluated for accident experience. If problems are occurring, however, they have not been noted.

CONCLUSIONS

The studies by Urbanik and Bonilla (*7*), Urbanik (unpublished research), and McCasland (*8,9*) indicate that the use of narrow shoulders and narrow lanes can be safe and cost-effective. This conclusion includes observation of long-term effects. The research suggests that left shoulder removals are preferred to right shoulder removals, because drivers prefer to use right shoulders. It is important to maintain at least one shoulder. Maintenance and enforcement personnel also prefer the retention of the right shoulder if a shoulder must be eliminated.

Limitations in the existing accident data base include the fact that most of the data are from Sun Belt states (Texas and California). Additional consideration must be given to snow removal and storage requirements when interpreting the data.

The most serious limitations in the current understanding of the use of shoulders and narrow lanes involve the questions of how narrow is too narrow and what are the compounding impacts of using several substandard design elements simultaneously? These issues apply to both lane width and shoulder width. For example, looking at left shoulder width it appears that, on the basis of experience, shoulders should be either 4 ft or less or 8 ft or more. Stated another way, shoulders wider than 4 ft and less than 8 ft give the appearance of being wide enough to park a car when in fact they are not. Although data have suggested two break points for left shoulders on the basis of empirical observation, minimum and desirable values for left shoulder widths have not clearly been demonstrated. Furthermore, the need to occasionally use minimum widths on the right shoulder for short distances (e.g., to avoid rebuilding or relocating bridge supports) requires careful consideration, because it is likely that the "shy" effect on the passenger side (right side) is greater than that on the driver side. That is to say, the data for clearances to the median barrier that have been provided should not be assumed to be valid for right side clearances.

Capacity and weaving do not appear to be adversely affected by reduced left shoulder width or by 11-ft lanes. There may be an impact of reduced left shoulder width on horizontal sight distance; however, the effects of this are unknown. Some existing facilities are known to possess sight distance deficiencies with no known documentation of problems.

Full left and right shoulders appear to be desirable features on all freeways. However, the removal of the left shoulder and the narrowing of lanes to 11 ft to increase capacity in a congested corridor may be an appropriate treatment when the only reasonable alternative is not to provide additional capacity. Traffic operations and traffic safety are improved on congested urban freeways when left shoulders are reduced and lanes are narrowed to 11 ft if the project is properly developed. Left shoulder removals are preferable to right shoulder removals because the right shoulder is usually selected by motorists when given a choice. The removal of both shoulders for any significant distance does not appear to be desirable from either an operational or safety viewpoint.

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Guidelines for Right-Turn Lanes on Urban Roadways

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Guidelines for the use of right-turn lanes at access points on urban two-lane and four-lane roadways were developed. The guidelines define the design-hour traffic volumes for which the benefits of right-turn lanes exceed their costs. The benefits used in the analysis were the operational and accident cost savings that right-turn lanes provide road users. The operational cost savings were those associated with the reductions in stops, delays, and fuel consumption experienced by through traffic. The accident cost savings were those associated with the reduction in accidents expected from the lower speed differentials between right-turning and through traffic. The guidelines define the right-turn design-hour volume required to justify a right-turn lane as a function of the following factors: (a) directional design-hour volume, (b) roadway speed, (c) number of lanes on the roadway, and (d) right-of-way cost. Comparison with guidelines developed by others indicates that the guidelines developed in the research are within the range of existing guidelines. In addition they are more definitive than the other guidelines because they account for the effects of roadway speed and right-of-way costs.

Right-turn movements from roadways can cause safety and operational problems. Vehicles slowing to turn right increase the potential for rear-end collisions involving the through vehicles following behind them that fail to slow down. It has been estimated that vehicles turning right into driveways account for 15 percent of all driveway accidents (*1*). About 7 percent of all traffic accidents in urban areas in Nebraska are collisions at driveways, and another 1 percent involve right-turn movements at intersections (*2*). Of course, the numbers of accidents related to right turns may be substantially underreported because some rear-end and sideswipe collisions occurring upstream of driveways and intersections as a result of right turns into them do not involve the right-turning vehicles. Vehicles slowing to turn right also increase the delay to through vehicles behind them and reduce the capacity of the highway. The delay experienced by the through traffic can range from a few seconds to over 20 sec per right turn, depending on the speed and volume of traffic (*3*). It has been estimated that the capacity of a four-lane arterial street with a 72-km/hr speed limit is reduced by 1 percent for every 2 percent of the traffic that turns right into driveways (*4*).

Right-turn lanes remove decelerating right-turn vehicles from the through lanes and thereby improve the safety and efficiency of traffic operations on the roadway. However, there are few guidelines available for determining when right-turn lanes should be provided at driveways and intersections on urban roadways. The national design guides do not include definitive warrants for right-turn lanes. AASHTO merely acknowledges in *A Policy on Geometric Design of Highways and Streets* (*5*) the potential benefits of right-turn lanes, particularly at intersections on high-speed, high-volume roadways, and suggests that the decision to provide right-turn lanes requires the consideration of several factors such as speeds, traffic volumes, capacity, type of highway, service provided, arrangement and frequency of intersections, and accident experience. Likewise, the ITE *Guidelines for Driveway Design and Location* (*6*) does not contain warrants for the use of right-turn lanes. Although the benefits of right-turn lanes are apparent, current nationally recognized highway design and access control guidelines do not define the prevailing roadway and traffic conditions for which these lanes are cost-effective on urban highways.

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OBJECTIVE

The objective of the research presented in this paper was to develop guidelines for the use of right-turn lanes on uncontrolled approaches to intersections and driveways on urban two-lane and four-lane roadways. The guidelines developed define the circumstances for which the costs of right-turn lanes are justified by the operational and accident cost savings they provide to road users.

EXISTING GUIDELINES

A few studies have developed guidelines for right-turn lanes. Alexander (*7*) developed warrants for right-turn lanes at intersections on two-lane highways solely on the basis of delay cost savings. Alexander compared the delay cost savings provided by right-turn lanes with the cost of constructing and maintaining them and identified the combinations of right-turn and approach volumes for which right-turn lanes would provide delay cost savings exceeding the cost of constructing and maintaining the right-turn lanes for average roadway speeds of 48, 64, and 81 km/hr.

The access control guidelines for urban streets and highways developed by Stover et al. (*3*) suggest that right-turn lanes be provided on uncontrolled intersection approaches when the average daily traffic (ADT) on the intersecting roadway is 500 vehicles per day (vpd) or greater. Right-turn lanes are also recommended at commercial and industrial driveways along primary and secondary streets. However these guidelines were simply based on a general assessment of the operational and safety effects of right-turn lanes with respect to the level of service implied by the functional classification of the streets. A benefit-cost analysis was not concluded.

Glennon et al. (*8*) conducted a benefit-cost analysis of right-turn deceleration lanes at driveway entrances. The analysis was based on data from the literature and some assumptions about the operational and safety effects of right-turn lanes. The results of the analysis indicated that right-turn lanes are cost-effective at driveways when (a) the driveway volume is at least 1,000 vpd with at least 40 right turns into the driveway during peak periods and (b) the roadway ADT is at least 10,000 vpd and the roadway speed is at least 56 km/hr.

Cottrell (*9*) developed guidelines for the treatment of right-turn movements at intersections on rural highways. The treatments considered were (a) no special treatment other than the radius, (b) a taper, and (c) a full-width lane. The guidelines were a synthesis of information obtained from a survey of state practices and field studies. Traffic conflict studies were conducted on 21 rural intersection approaches in Virginia in an effort to determine the relationship between right-turn conflicts, traffic volume, and type of right-turn treatment. Right-turn lanes were found to reduce right-turn conflicts, but the data were not sufficient for the development of guidelines. Therefore, the guidelines developed by Cottrell (*9*) were a synthesis of other states' guidelines adjusted to reflect the nature of traffic conditions in Virginia as determined from the field studies. Similar guidelines have been adopted by the state of Washington (*10*).

Stover and Koepke (*11*) recommend driveway designs for access to arterial streets that include right-turn lanes. They suggest that continuous right-turn lanes be provided when the driveway spacing is less than that necessary to accommodate right-turn lanes at individual driveways. Also, on streets where the speed is over 56 km/hr, they recommend the use of right-turn lanes at driveways when there are more than 1,000 right turns per day and 40 right turns during the peak hour.

In a national study of roadway access management practices, Koepke and Levinson (*12*) cite the right-turn lane warrant used by the Colorado Department of Transportation. The warrant recommends the provision of right-turn deceleration lanes at access points on the basis of the right-turn volume, the roadway's single-lane volume, and roadway speed.

The existing guidelines use several factors to determine the need for right-turn lanes, such as right-turn and through traffic volumes, traffic speed, roadway classification, number of roadway lanes, and capacity. Although many of the guidelines use the same factors, there is considerable variation among the threshold values and the units applied to them. For example, traffic volumes are expressed in terms of ADTs in some of the guidelines, design-hour volumes in others, and peak-hour volumes in others. Some of the guidelines use right-turn volumes and some use right-turn percentages. Some of the guidelines are based primarily on experience and engineering judgment, whereas others are based on benefit-cost analyses. Even among those based on benefit-cost analyses, however, different benefits and costs were used in the analyses. Both operational and safety benefits were used in some cases, whereas only operational benefits were included in others. None of the guidelines have been widely adopted by practitioners.

OPERATIONAL EFFECTS

Right-turn lanes remove the decelerating right-turning vehicles from the through traffic lanes and thereby eliminate the need for through traffic to slow down or change lanes behind them. Consequently, right-turn lanes improve the operational efficiency of the roadway by eliminating the through-vehicle delay and operating costs associated with the speed-change cycle. To quantify these operational improvements, the TRAF-NETSIM (*13*) model was used to simulate traffic operations on uncontrolled approaches to intersections and driveways with and without right-turn lanes. Multiple regression analysis of the simulation output was then conducted to derive equations for the operational benefits of right-turn lanes.

Simulation

The TRAF-NETSIM model was used to simulate traffic operations at four T-intersection configurations: (a) uncontrolled intersection approach without a right-turn lane on a two-lane, two-way roadway, (b) uncontrolled intersection approach with a right-turn lane on a two-lane, two-way roadway, (c) uncontrolled intersection approach without a right-turn lane on a four-lane, two-way roadway, and (d) uncontrolled intersection approach with a right-turn lane on a four-lane, two-way roadway. In each case the intersecting roadway was a two-lane, two-way roadway that was controlled by a stop sign.

Link-Node Diagram

The link-node diagram used to represent the four intersection configurations is shown in Figure 1. Node 4 is the intersection. The roadway is represented by the links between Nodes 801, 1, 4, 2, and 802. Link 1–4 is the uncontrolled intersection approach that was simulated with and without a right-turn lane. The intersecting roadway or driveway is represented by the links between Nodes 803, 3, and 4. Link 3–4 is controlled by a stop sign. The roadway links each had one lane when operations on a two-lane, two-way roadway were being simulated, and they each had two lanes when a four-lane roadway was being simulated. The links on the intersecting roadway or driveway each had one lane in all cases. The TRAF-NETSIM performance measures output for Links 1–4 and 4–2 before and after the addition of a right-turn lane on Link 1–4 were compared to determine the operational effects of the right-turn lanes.

Experimental Design

The inventory of urban highways maintained by the Nebraska Department of Roads (NDOR) was reviewed to determine the

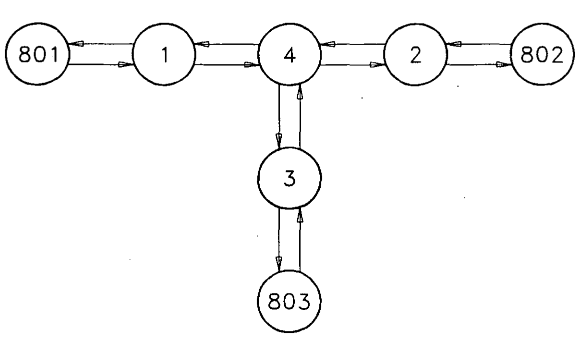

FIGURE 1 Link-node diagram.

range in traffic volumes that should be considered in the simulation. The ADTs on two-lane sections ranged from around 1,000 to 25,000 vpd. The ADTs on four-lane sections ranged from about 5,000 to 60,000 vpd. Since ADTs are two-way volumes and the peak-hour traffic represents about 10 percent of the ADT (*14*), the link volumes used in the simulation ranged from 100 to 1,200 vehicles per hour (vph) for the two-lane, two-way roadway and from 600 to 3,000 vph for the four-lane, two-way roadway. Zero percent trucks was used in all cases, because the effects of trucks were accounted for in the road user cost factors used in subsequent benefit-cost analysis.

Data defining the range of the right-turn percentages at driveways on urban highways in Nebraska were not available. Of course the percentage of right turns would depend on the nature and intensity of the abutting land use and would vary by time of day. For example driveways serving an office building may generate higher right-turn percentages during the morning peak hours, whereas shopping center driveways may generate higher right-turn percentages during off-peak hours. Therefore to maximize the applicability of the guidelines to be developed, a wide range of right-turn percentages was used in the simulation. The right-turn percentages simulated ranged from 7.5 to 90 percent at 7.5-percent increments.

Four roadway speeds and three driveway speeds were simulated. The roadway speeds were 40, 56, 72, and 89 km/hr. The driveway speeds were 16, 24, and 32 km/hr.

A total of 8,640 simulations were made. Three runs were made for each combination of the following variables:

1. Number of roadway lanes (two levels: two and four lanes),
2. Right-turn lane (two levels: with and without),
3. Approach volume (five levels),
4. Right-turn percentage (12 levels: 7.5 to 95 percent at 7.5-percent increments),
5. Approach speed (four levels: 40, 56, 72, and 89 km/hr), and
6. Driveway speed (three levels: 16, 24, and 32 km/hr).

The five levels of approach volume simulated for two-lane roadways were 100, 300, 600, 900, and 1,200 vph. For four-lane roadways, they were 600, 1,200, 1,800, 2,400, and 3,000 vph.

Data Analysis

The delay, stops, and fuel consumption for the through vehicles simulated by TRAF-NETSIM for Links 1–4 and 4–2 shown in Figure 1 were recorded from the output for each simulation. For the conditions simulated it was found that right-turning vehicles on roadways without right-turn lanes caused through vehicles to slow but not stop. Therefore there were no differences in the number of through-vehicle stops with and without right-turn lanes. Consequently the only operational effects of right-turn lanes obtained from the results of the simulation were reductions in delay and fuel consumption.

A total of 8,640 delay and fuel consumption values were obtained from the TRAF-NETSIM output, 4,320 for each roadway type. The data for each roadway type were then split randomly into two sets. One set, which contained two-thirds of the original data, was used to conduct a multiple regression analysis to develop delay and fuel consumption models. The other set, which contained one-third of the original data, was used to validate the models.

Model Formulation

The first step in the formulation of the delay and fuel consumption models was to determine the nature of the relationships. Scattered diagrams were plotted with delay and fuel consumption as the dependent variables and volume as the independent variable. Diagrams were plotted for various combinations of right-turn percentage, roadway speed, and driveway speed for two-lane and four-lane roadways with and without right-turn lanes. Examination of these diagrams suggested the nature of the functional relationships that should be investigated in the multiple regression analysis. Linear relationships were indicated for the fuel consumption models. Both linear and exponential relationships were indicated for the delay models.

Next multiple regression analysis was used to develop delay and fuel consumption models for both the two-lane and the four-lane roadways. Several models were considered. The alternative models were compared on the basis of the following: (a) the extent to which they explained the variation in the dependent variable, as indicated by their coefficients of determination (R^2 values); (b) the statistical significance of the independent variables, as indicated by their "F" values; (c) the extent to which they exhibited the lack of multicollinearity, as indicated by their variance inflation factors; and (d) their appropriateness, as indicated by their residual plots.

Four linear models were found to best describe the delay and fuel consumption on two-lane and four-lane roadways. The coefficients of determination were 0.77 and 0.80 for the two-lane and four-lane delay models, respectively, and 0.99 for both of the fuel consumption models. All four of the models were statistically significant (p = .0001), and the regression coefficients of all the independent variables in the models were also statistically significant (p = .0001). The residual plots indicated that the variance of the error terms was constant, indicating that the relationships were appropriate.

According to the models right-turn lanes reduced delay and fuel consumption as a function of right-turn volume as follows:

$$\Delta D_{2L} = 0.0388\ V_{RT} \quad (1)$$

$$\Delta FC_{2L} = 0.0125\ V_{RT} \quad (2)$$

$$\Delta D_{4L} = 0.0200\ V_{RT} \quad (3)$$

$$\Delta FC_{4L} = 0.00435\ V_{RT} \quad (4)$$

where

ΔD_{2L} = delay savings on a two-lane roadway (sec/through vehicle),
ΔFC_{2L} = fuel consumption savings on a two-lane roadway (L/15 min),
ΔD_{4L} = delay savings on a four-lane roadway (sec/through vehicle),
ΔFC_{4L} = fuel consumption savings on a four-lane roadway (L/15 min), and
V_{RT} = right-turn volume (vehicles/15 min).

Model Validation

The delay and fuel consumption models were validated by conducting a multiple regression analysis of the data that were set aside for model validation and comparing the results with those of the initial regression analysis. A comparison of the results of the initial and validation regression analyses indicated that there was no statistically significant ($p = .01$) difference between the regression coefficients obtained from the two analyses. Therefore it was concluded that the delay and fuel consumption models developed initially are valid for the purpose of the study.

SAFETY EFFECTS

Right-turn lanes improve the safety of traffic operations by removing the deceleration of right-turning vehicles from through traffic lanes, thereby reducing the potential for rear-end collisions involving through vehicles that fail to slow down. Previous research, however, has not adequately quantified the safety effects of right-turn lanes on uncontrolled approaches to intersections and driveways on urban roadways because of the limitations of the accident data available. Therefore as suggested by Stover et al. (*3*), the relationship between speed differential and accidents established by Solomon (*15*) was used to estimate the safety effects of right-turn lanes, because the primary effect of a right-turn lane is to reduce the speed differential in the through lanes.

Speed Differential

To estimate accidents from the relationship between speed differential and accidents established by Solomon (*15*), it was first necessary to estimate the difference between the speed of right-turning vehicles and the average speed of vehicles on the roadway. The average speed of right-turning vehicles during deceleration was then assumed to be equal to the average of the speeds of the vehicles on the roadway and driveway entrance. On the basis of the driveway entrance speed data collected by Richards (*16*) and Stover et al. (*3*), it was assumed that the turning speed of right-turning vehicles is 24 km/hr.

The average roadway speed is the average speed of all vehicles on the roadway, both right-turning and non-right-turning vehicles. The average roadway speed was computed as follows:

$$S_{avg} = P_{RT}S_{RT} + (1 - P_{RT})S_R \qquad (5)$$

where

S_{avg} = average roadway speed (km/hr),
P_{RT} = portion of right-turning vehicles,
S_{RT} = average speed of right-turning vehicles (km/hr), and
S_R = roadway speed (km/hr).

The speed differentials used to estimate the accidents associated with right turns from through lanes were the differences between the average speeds of right-turning vehicles and the average roadway speeds.

Number of Accidents

The number of accidents per year caused by vehicles turning right from through traffic lanes was computed as follows:

$$A = 55.6(P_D I_D + P_N I_N)\ \mathrm{ADT} \cdot P_{RT} \cdot L \qquad (6)$$

where

A = annual number of accidents caused by right-turning vehicles,
P_D = portion of daytime traffic,
I_D = daytime accident involvement rate (accidents/vehicle km),
P_N = portion of nighttime traffic,
I_N = nighttime accident involvement rate (accidents/vehicle km),
ADT = annual average daily traffic (vpd),
P_{RT} = portion of right turns, and
L = right-turn deceleration distance (m).

The daytime and nighttime accident involvement rates were determined from the relationship between speed differential and accidents established by Solomon (*15*). The portions of daytime and nighttime traffic used in Equation 6 were the averages of those found at the continuous traffic counting stations on urban arterial sections of the state highway system in Nebraska (*14*). On average the portion of daytime traffic is 0.76 and the portion of nighttime traffic is 0.24. The right-turn deceleration distance in Equation 6 is the distance over which the right-turning vehicles are assumed to decelerate, and the length of the roadway over which the speed differential used to determine the accident involvement rate was assumed to apply. The deceleration distances used are those recommended by AASHTO (*5*).

BENEFITS-COSTS

The development of the right-turn lane guidelines was based on the results of a benefit-cost analysis. The benefits used in the analysis were the road user cost savings associated with the operational and safety effects of right-turn lanes. The costs used in the analysis were those of constructing and maintaining right-turn lanes.

Operational Cost Savings

The operational cost savings were the road user cost savings resulting from the reductions in delay and fuel consumption provided by right-turn lanes. Using the delay and fuel consumption savings models (Equations 1 to 4), the hourly operational cost savings associated with these savings in delay and fuel consumption are computed as follows:

$$\mathrm{HOCS}_{2L} = \frac{0.0338}{3600} V_{RT}V_T C_T + (0.0125)(4)V_{RT}C_F \qquad (7)$$

$$\mathrm{HOCS}_{4L} = \frac{0.0200}{3600} V_{RT}V_T C_T + (0.00435)(4)V_{RT}C_F \qquad (8)$$

where

HOCS_{2L} = hourly operational cost savings on a two-lane roadway ($/hr),
HOCS_{4L} = hourly operational cost savings on a four-lane roadway ($/hr),
V_{RT} = right-turn volume (vehicles/15 min),
V_T = through traffic volume (vph),

C_T = unit value of time (\$/hr), and
C_F = cost of fuel (\$/L).

To facilitate the ultimate application of the guidelines to be developed, the through and right-turn volumes in Equations 7 and 8 were expressed in terms of ADTs as follows:

$$V_{RT} = P_{RT} \frac{P_i}{4} \frac{ADT}{2} \tag{9}$$

$$V_T = (1 - P_{RT}) P_i \frac{ADT}{2} \tag{10}$$

where

V_{RT} = right-turn volume (vehicles/15 min),
V_T = through traffic volume (vph),
P_{RT} = portion of right turns,
P_i = portion of ADT in ith hour of the day, and
ADT = annual average daily traffic (vpd).

Substituting these volume expressions into Equations 7 and 8, the hourly operational cost savings equations in terms of ADT become:

$$\text{HOCS}_{2L} = \left[\frac{0.0338}{57{,}600} (1 - P_{RT})P_i\text{ADT } C_T + \frac{0.0125}{2} C_F\right] P_{RT}P_i\text{ADT} \tag{11}$$

$$\text{HOCS}_{4L} = \left[\frac{0.0200}{57{,}600} (1 - P_{RT})P_i\text{ADT } C_T + \frac{0.00435}{2} C_F\right] P_{RT}P_i\text{ADT} \tag{12}$$

The portion, P_i, of daily traffic during each hour of the day used to compute the operational cost savings was determined from the traffic count data collected at the continuous traffic counting stations on urban arterial sections of the state highway system in Nebraska (*14*).

The annual operational cost savings were then computed by summing the hourly operational cost savings for each of the 24 hr in the day to obtain the daily operational cost savings and then multiplying the daily operational cost savings by 365 days per year.

The unit value of time, C_T, used to compute the operational cost savings was \$9.53/hr. This value is the 1975 unit value of time established by AASHTO (*17*) updated to 1992 in accordance with changes in the consumer price index. This value represents an average unit value of time for all trip purposes, relatively low (less than 5 min) time savings, an average vehicle occupancy of 1.56 persons, and a vehicle mix of 97 percent passenger cars, 2 percent single-unit trucks, and 1 percent combination trucks. The vehicle mix was the average composition of traffic at the continuous traffic counting stations on urban arterial sections of the state highway system in Nebraska (*14*). The cost of fuel used in the calculation of operation cost savings was \$0.32/L.

Accident Cost Savings

The number of accidents per year caused by vehicles turning right from through-traffic lanes was computed by using Equation 6. This number represents the number of rear-end accidents likely to be caused by vehicles decelerating to turn right from through-traffic lanes. Therefore, it was assumed that this number of accidents would be eliminated by providing a right-turn lane.

According to the NDOR revised relative severity index figures (*18*), the cost of a rear-end collision on an urban section of the state highway system in Nebraska is \$9,300. Thus the annual accident cost savings provided by right-turn lanes were computed as follows:

$$\text{AACS} = \$9{,}300A \tag{13}$$

where AACS is the annual accident cost savings (\$/year), and A is the annual number of accidents caused by right-turning vehicles.

Right-Turn Lane Costs

The costs of right-turn lanes were estimated from cost data provided by the NDOR for the construction of right-turn lanes on typical urban sections of the state highway system in Nebraska. The costs included fixed costs, variable costs, and right-of-way cost. The fixed costs included the costs of preliminary engineering, mobilization, field laboratory, general clearing and grubbing, and traffic control devices. The variable costs were a function of the pavement area of the right-turn lane and included the costs of excavation, paving, sodding, and sidewalks. The total variable cost was computed by multiplying the pavement area of the right-turn lane by the unit variable cost. The area of the right-turn lane was calculated by using a width of 3.66 m and a length that included the approach-taper deceleration-lane distances recommended by AASHTO (*5*).

The cost of right-of-way can vary considerably. The experience of the NDOR indicates that it can range from \$0.093 to \$0.93/m^2 along urban roadways, depending on the location. Also in some cases there may be no cost of right-of-way because the existing right-of-way is sufficient to accommodate the construction of a right-turn lane. Therefore the guidelines were developed for four cases. One case represented the situation in which the existing right-of-way was sufficient so that the right-of-way cost was zero. The other three cases were representative of low (\$0.093/m^2), medium (\$0.465/m^2), and high (\$0.93/m^2) right-of-way costs.

GUIDELINES

The guidelines were developed by comparing the benefits and costs of right-turn lanes at uncontrolled intersections and driveways on urban roadways. The guidelines indicate the design-hour traffic volumes for which the costs of right-turn lanes are justified by the benefits they provide to road users. The benefits and costs of right-turn lanes on two-lane and four-lane roadways were compared over a range of traffic volumes. The benefits used were the sum of the annual operational and accident costs savings, which were computed for annual traffic growth rates of 2 percent. The costs of right-turn lanes were annualized by using a 4-percent interest rate, a 20-year service life, and a zero residual or salvage value. Annual costs were computed for the four right-of-way cost cases: (a) none, construction within existing right-of-way; (b) low, \$0.093/m^2; (c) medium, \$0.465/m^2; and (d) \$0.93/m^2.

The ADTs and right-turn percentages at which the benefits and costs of right-turn lanes are equal were determined for each combination of roadway speed and right-of-way cost. The breakeven ADTs and right-turn percentages were then converted to design-hour volumes by using the relationship between design-hour volume and ADT on urban roadways in Nebraska (*14*) as follows:

$$\mathrm{DDHV} = 65.11 + 0.0958\,\frac{\mathrm{ADT}}{2} \tag{14}$$

$$\mathrm{RTDHV} = P_{\mathrm{RT}}\mathrm{DDHV} \tag{15}$$

where

DDHV = directional design hour volume (vph),
RTDHV = right-turn design hour volume (vph), and
ADT = annual average daily traffic (vpd).

RTDHV represents the minimum design-hour right-turn volume necessary to justify the construction of a right-turn lane on an urban roadway with a directional design-hour volume equal to DDHV.

The guidelines for right-turn lanes on urban two-lane roadways are given in Table 1. The guidelines for right-turn lanes on urban four-lane roadways are given in Table 2. In each case, guidelines are shown for each combination of roadway speed and right-of-way cost. It should be noted that both the directional and right-turn design-hour volumes in the guidelines are existing, or base year, traffic volumes.

The guidelines developed in this research are within the range of those developed by others. The guidelines for right-turn (RT) lanes on urban two-lane and four-lane roadways are compared with the right-turn lane guidelines developed by others in Figures 2 and 3, respectively. The ranges of the guidelines developed in this research are defined by two cases. The upper limits of the ranges are defined by the guidelines for the 40-km/hr roadway speed and high ($0.93/m^2) right-of-way cost, and the lower limits are defined by the guidelines for the 89-km/hr roadway speed and zero right-of-way cost. As shown in Figures 2 and 3, the guidelines developed in this research are bounded by the existing guidelines. The Colorado (*12*) guidelines are the lower boundary, and the Virginia (*9*) and Washington State Department of Transportation (*10*) guidelines are the upper boundary. The guidelines developed by Alexander (*7*) and Glennon et al. (*8*) are within the range of those developed in the present research.

CONCLUSION

The guidelines presented in this paper define the right-turn design-hour volume required to justify a right-turn lane at access points on urban two-lane and four-lane roadways as a function of the following factors: (a) directional design-hour volume, (b) roadway speed, (c) number of lanes on the roadway, and (d) right-of-way cost. The guidelines indicate that the right-turn design-hour volume that warrants a right-turn lane is lower on roadways with higher directional design-hour volumes and higher roadway speeds, because the road user costs associated with the operational and safety effects of right turns are greater on higher-volume, higher-speed roadways. Consequently the road user cost savings provided by right-turn lanes are greater on these roadways. Likewise the right-turn design-hour volume required to justify a right-turn lane on a two-lane roadway is lower than that required to

TABLE 1 Right-Turn Lane Guidelines for Urban Two-Lane Roadways

Roadway DDHV (vph)	Minimum Right-Turn DHV (vph)															
	Within Existing ROW				ROW Cost = $0.093/m^2				ROW Cost = $0.465/m^2				ROW Cost = $0.93/m^2			
	Roadway Speed (km/hr)				Roadway Speed (km/hr)				Roadway Speed (km/hr)				Roadway Speed (km/hr)			
	40	56	72	89	40	56	72	89	40	56	72	89	40	56	72	89
100			65	30			70	40								
125	65	60	40	25	70	65	50	25			75	45				
150	60	50	35	20	65	55	40	20	75	75	60	35	95	95	90	50
200	50	45	30	15	55	45	30	15	65	65	40	25	80	80	60	30
400	40	35	20	10	40	35	20	10	40	40	30	20	55	55	40	20
600	35	30	15	10	35	30	15	10	35	35	25	15	45	45	35	15
800	30	25	15	10	30	25	15	10	30	30	20	10	35	35	30	15
1000	25	20	15	10	30	25	15	10	30	30	20	10	35	35	30	15
1200	25	20	15	10	30	25	15	10	30	30	20	10	35	35	30	15

TABLE 2 Right-Turn Lane Guidelines for Urban Four-Lane Roadways

Roadway DDHV (vph)	Minimum Right-Turn DHV (vph)															
	Within Existing ROW				ROW Cost = $0.093/m²				ROW Cost = $0.465/m²				ROW Cost = $0.93/m²			
	Roadway Speed (km/hr)				Roadway Speed (km/hr)				Roadway Speed (km/hr)				Roadway Speed (km/hr)			
	40	56	72	89	40	56	72	89	40	56	72	89	40	56	72	89
100				35				60								
150	80	65	40	25	85	70	45	25			70	40				60
200	70	55	35	20	75	60	35	20	85	75	50	30	110	100	70	40
500	45	40	25	15	50	45	25	15	60	50	35	25	70	60	40	30
1000	35	30	20	10	35	30	20	10	40	40	25	15	45	45	35	20
1500	30	25	15	5	30	25	15	5	35	35	20	10	40	40	30	15
2000	25	20	15	5	25	20	15	5	30	30	20	10	35	35	25	15
2500	20	20	15	5	20	20	15	5	25	25	20	10	30	30	20	15
3000	20	20	15	5	20	20	15	5	25	25	20	10	25	25	20	15

justify one on a four-lane roadway, because the road user costs associated with the operational and safety effects of right turns are higher on two-lane roadways. On the other hand, the warranting right-turn design-hour volume increases with higher right-of-way cost, because more road user cost savings are needed to offset the higher cost of the right-turn lane.

The guidelines developed in this research are within the range of those developed by others. Comparison with other guidelines indicates that the guidelines developed in this research are reasonable. In addition, they are more definitive than the other guidelines because they account for the effects of roadway speed and right-of-way costs.

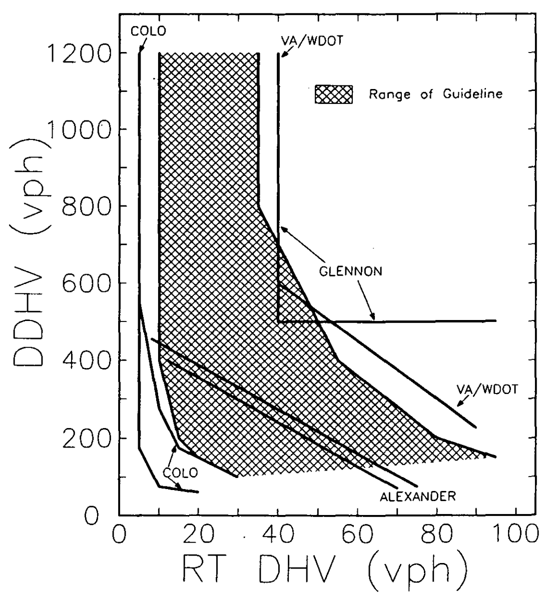

FIGURE 2 Comparison of right-turn lane guidelines for urban two-lane roadways.

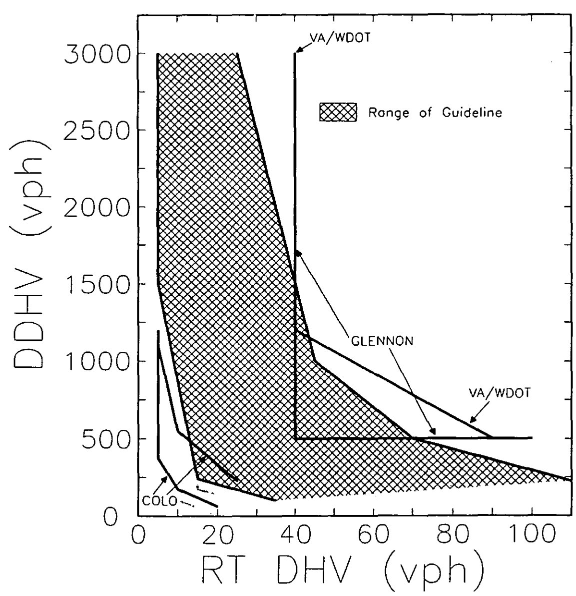

FIGURE 3 Comparison of right-turn lane guidelines for urban four-lane roadways.

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Evaluating the Quality of Cities' Geometric Design Standards

J. L. GATTIS AND M. V. KALYANAPURAM

Are cities employing state-of-the-practice street design standards? An evaluation of the street design standards used by medium-size and large cities in Oklahoma is presented. The researchers asked city staff in 19 cities to send their various geometric design and subdivision standards, and staff in 12 of those cities responded. The researchers evaluated stopping sight distance, horizontal curvature, gradient, street section width, and intersection radius standards. The standards evaluated were those intended for new developments or streets. The researchers established "recommended" design practices by referring to nationally recognized publications. The standards of each city were compared with the recommended practices to determine the adequacy of the city design standards, and the city standards were evaluated on the basis of a system devised by the researchers. The evaluation results indicated that the 12 cities had good gradient standards. Turning radius standards were generally adequate, and the adequacy of stopping sight distance standards was mixed. The standards for centerline radius and for arterial street section widths were often inadequate. The need to improve the quality of street design standards at the local level was also discussed by the researchers. The 1991 federal transportation bill (Intermodal Surface Transportation Efficiency Act) mandates a review of state standards for highways that receive federal aid; it is suggested that an outside review of city design standards may be needed.

The general public has become more aware of quality in everything from consumer products to medical services. When a profession or an industry fails to mandate quality (i.e., high standards), the public reacts negatively. Current public attitudes toward lawyers and politicians are cases in point.

Transportation engineers define quality in a number of ways, including the adequacy of roadway design criteria. Much effort has gone into preparing *A Policy on Geometric Design of Highways and Streets*, or the Green Book (*1*), and other publications so engineers can follow state-of-the-practice design criteria. State departments of transportation attempt to employ adequate design practices, and FHWA requires projects to be designed to comply with good practices. On the other hand, city streets are designed under the auspices of local governments, which have neither the federal oversight nor the vast resources of a state agency to effect adherence to accepted good practices. Do the cities have state-of-the-practice standards, or are city street design standards inadequate?

To evaluate the quality or adequacy of the geometric design criteria used by the cities in one state, the researchers evaluated the street design standards used by large and medium-size cities in Oklahoma. Staff in 12 of the 19 cities contacted sent their various geometric design and subdivision standards to the researchers. The researchers evaluated stopping sight distance, horizontal curvature, gradient, section width, and intersection radius standards. The standards evaluated were those intended for new streets. The researchers established "recommended" design practices after referring to nationally recognized publications. The standards of each city were compared with the recommended practices to determine the quality of the city design standards, and the city standards were evaluated on the basis of a system devised by the researchers.

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BACKGROUND

The street system forms the framework for community development and permits the circulation of people and goods throughout the community. A city needs good design criteria to develop a functionally efficient and safe street system. Progressive design criteria permit citizens to get quality streets from their tax dollars. A poorly designed road gives an inferior level of service to the traveling public and can create conditions conducive to accidents and tort lawsuits.

To establish a set of geometric design standards that is both adequate and comprehensive, the engineer must have a good understanding of underlying design fundamentals. These fundamentals include an appreciation of interactions between the driver, the vehicle, and the roadway. The engineer must also appreciate the limitations of the driver and the vehicle and recognize that the roadway should accommodate the limitations of prudent drivers and vehicles that are not defective.

State and national agencies have the size and funding to employ civil engineers specializing in transportation engineering. These transportation engineers can draw from their own and others' experiences, continuing education opportunities, and other resources to maintain a knowledge of current street design issues.

In contrast many local governments employ only a few civil engineers. These engineers may be responsible for water, wastewater, solid waste, storm water and waterways, public structures, as well as street design and traffic control devices. It is difficult for an engineer to be an expert in all of these areas; many cities employ no engineers with transportation expertise. The local engineer may be subject to direct pressure from city councilmembers (who neither know nor appreciate fundamental geometric design concepts) to accommodate developers, emotional citizens, or other pressure groups. Local politicians and engineers may feel pressure to stretch the funds used for street paving past the limits of accepted design practices. Any of these factors may create an environment that is not conducive to the development and enforcement of state-of-the-practice geometric design standards at the local level.

RESEARCH STEPS

To evaluate city street standards, the researchers collected city design standards and reviewed them. The researchers created procedures to evaluate the adequacy of the standards.

Collecting the Data

The researchers solicited various geometric design and subdivision standards from 19 cities listed in the Oklahoma Municipal League directory (*2*) as having populations of 20,000 or more. Officials in 12 cities responded by providing design manuals, subdivision regulations, or detailed drawings of their street designs. The materials received were researched thoroughly, and a data base in which to store quantitative information was set up.

After analyzing the data, the researchers made field visits to each of the responding cities and administered a questionnaire to city staff. This helped the researchers understand city practices.

Analyzing the Data

The researchers established "recommended" design criteria by consulting publications such as the Green Book (*1*) and *Traffic Engineering Handbook* (*3*). The researchers also referred to *Residential Streets* (*4*) by ASCE, *Residential Street Design and Traffic Control* (*5*) by ITE, and *NCHRP Report 330* (*6*).

The standards of each city were compared with the recommended practices to determine the quality of various city design standards. To make a quantitative comparison, the researchers devised methods to evaluate how close each city came to following the recommended practices. The method assigned a rank of 1 to a particular city's standard if it appeared to meet or exceed recommended practices, a rank of 3 if the city's standard was less than recommended and somewhat marginal, and a rank of 5 if the city's standard was deficient. For some of the design topics, the 3 rank was further divided into ranks 2 and 4 to differentiate among degrees of adequacy. The details of the evaluation method are described in the following sections.

Central Concepts

The concepts of functional design and design speed were central to the analyses.

The functional design concept defines and differentiates among streets, depending on the degree to which a street provides property access or provides movement for higher volumes at higher speeds. There are three main functional classes of urban streets: arterials, collectors, and locals. For a given city a separate evaluation was made of the standards used for each functional class. If a city had both major arterial and minor arterial classes, then standards for the two classes were analyzed separately.

Design speed is an important roadway design control. The chosen design speed must be high enough to accommodate the expectations of most drivers who will use the road. The design speed for most facilities is the one at which 85 to 90 percent of the users drive (*1*). Good practice dictates that various roadway elements —alignment, sign placement, and intersection spacing—must accommodate drivers traveling at the design speed. One basis for the evaluations was how well various design elements accommodated vehicles traveling at design speed.

DATA ANALYSIS

When cataloging and analyzing data, the researchers kept the identities of the cities confidential by using letters in place of the city names. The 12 cities were referred to as A, B, C, D, E, F, G, H, J, K, L, and M.

The researchers evaluated stopping sight distance, horizontal curvature, gradient, section width, and intersection radius standards. These standards were chosen for evaluation because most of the cities furnished information with which to evaluate these items and because relatively objective criteria exist for these items.

Stopping Sight Distance

There should be enough sight distance available on the roadway so that a driver in a vehicle traveling at the design speed can stop before reaching a stationary object ahead (*1*). Stopping sight distance (SSD) in feet is calculated from the formula

$$SSD = 1.467Vt + V^2/[30\ (f + g)] \qquad (1)$$

where

V = initial speed, mph,
f = coefficient of friction,
t = 2.5 sec, and
g = decimal grade.

The adequacy of each city's SSD standards was evaluated by comparing, for each functional class, the city's design speed with the speed for which the city's SSD was adequate. The researchers developed a statistical methodology to evaluate the adequacy of the city standard. The analysis incorporated the following assumptions:

- Vehicle speeds follow a normal distribution;
- Ninety percent of drivers will travel at or less than the design speed; and
- Standard deviation (σ) of the speed distribution was 5 mph.

For those cities that listed no design speeds in the documents sent to the researchers, the following design speeds were assumed:

- Arterial or major arterial, 40 mph;
- Minor arterial, 35 mph;
- Collector, 30 mph; and
- Local residential, 25 mph.

With the stated assumptions, 90 percent of drivers traveled at or less than the design speed, and the median speed was approximated to be 6.41 mph less (i.e., $1.282 * \sigma = 6.41$) than the design speed.

For each city and functional class, the researchers calculated the maximum speed for which the city standard for SSD was safe. The maximum safe speed (i.e., the speed accommodated by the city standard for SSD) was then compared with the design speed.

The researchers assumed a level gradient, which yields a less rigorous criterion than that which could have been applied. With the assumed median speed and standard deviation, the researchers calculated the percentage of the drivers who were afforded adequate SSD as they traveled streets designed to meet the city standard.

If the city standard for SSD accommodated the speeds of 90 percent or more of the drivers, the city standard got 1 as its rank. If the city standard accommodated between 89.9 and 80.0 percent, the city standard got a rank of 2. A percentage of between 79.9 and 70.0 got a rank of 3, and a rank of 4 was given if the percent accommodated fell between 69.9 and 60.0. Anything less than 60.0 percent fetched the lowest rank, 5, for the city standard. Table 1 gives this ranking system.

To illustrate the method City D had local residential street design speed (30 mph) and stopping sight distance (175 ft) standards. The 30-mph design speed was also the assumed 90th percentile speed, and the standard deviation was 5 mph. For an SSD of 175 ft the maximum safe speed was 28.0 mph.

$$\Delta = v - v_{\text{MED}} = v - (v_{\text{DES}} - 1.28\,\sigma)$$
$$= 28.0 - (30 - 6.41) = 4.41 \text{ mph} \quad (2)$$

$$\Delta/\sigma = 4.41/5.0 = 0.882 \quad (3)$$

For $Z = 0.882$ the area under the normal curve is 0.31 (two-tail) or 0.81 (one-tail). With the design speed as the 90th percentile benchmark, 81 percent of the drivers' speeds were accommodated by City D's SSD design standard, and City D received a 2 as the adequacy of its SSD design standard for local streets.

Although these evaluation assumptions were arbitrary, they were not unreasonable. The assumptions helped measure how close the cities came to following the recommended practices that were based on state-of-the-practice criteria. The ranking method caused any design element that was not even adequate for speeds 5.2 mph less than the design speed to get the lowest ranking. Table III-1 (wet pavement SSD) in the Green Book (*1*) lists a range of assumed speeds for each design speed; this range has 5-mph spread for the 45-mph design speed and a smaller spread for lower speeds. So for design speeds under 50 mph, the city standard would have to be below that needed to meet the lower SSD design values in Table III-1 of the Green Book (*1*) before the city would get a 5 rank.

This analysis was not performed on those cities that did not list sight distance standards. Table 2 lists the city standard design speed, the city standard for SSD, and the calculated speed for which the standard SSD was safe.

Horizontal Centerline Radius

Good horizontal alignment of a roadway requires that the road be laid out so that its curves are in a harmonious relationship with design speed, superelevation, and side friction. The equation to find a radius (R) suitable for a given combination of speed, cross fall (cross slope), and side friction is (*1*)

$$R_{\min} = \frac{V^2}{15(e + f)} \quad (4)$$

where

R = curve radius (ft),
V = vehicle speed (mph),
e = rate of superelevation (ft/ft), and
f = side friction factor.

For low-speed urban streets, the "urban" side friction values (*1*) were used.

In addition, there must be adequate SSD to "see around the curve up ahead." The needed sight line is a chord to the curve of the inside lane centerline, and the sight distance is measured along the inside lane centerline. This sight distance is found from the equation

$$\text{SSD} = \arccos(1 - M/R) * R/28.648 \quad (5)$$

where M is the offset distance in feet from the curve to the line of sight.

The maximum safe speed for a given radius is the least of the following three cases:

1. Maximum safe speed when the roadway has a positive cross fall,
2. Maximum safe speed when the roadway has a negative cross fall, or
3. Maximum safe speed for the available SSD around a curve.

Sometimes a city street is divided by a median and both roadways are superelevated with the curve, or sometimes an undivided city street will be superelevated across the entire cross section. More often, the outside of the curve will have a negative or an adverse cross fall, so in most cases the lesser of Case 2 or Case 3 is the critical situation.

The same assumptions, statistical analysis method, and ranking system used for the SSD analysis were used for analyzing the

TABLE 1 Ranges for Ranking City Standards

Number of standard deviations above median	Speed (mph) above assumed median	Speed (mph) below the design speed	Percent % accommodated by the design element	Rank
> 1.282	> 6.41	0	≥ 90.0	1
1.282 - 0.842	6.41 - 4.21	0.1 - 2.2	89.9 - 80.0	2
0.842 - 0.524	4.21 - 2.62	2.2 - 3.8	79.9 - 70.0	3
0.524 - 0.253	2.62 - 1.27	3.8 - 5.1	69.9 - 60.0	4
< 0.253	< 1.27	< 5.1	< 60.0	5

TABLE 2 Evaluation of SSD Standards

CITY	CITY DESIGN SPEEDS				CITY STOP SIGHT DIST				MAXIMUM SAFE SPEED, GIVEN CITY SSD			
	Maj. Art. mph	Min. Art. mph	Col. mph	Loc. Res. mph	Maj. Art. ft.	Min. Art. ft.	Col. ft.	Loc. Res. ft.	Maj. Art. mph	Min. Art. mph	Col. mph	Loc. Res. mph
A					300		250	200	39.0		35.2	30.5
B												
C												
D	60	30	35	30	350	200	250	175	42.7	30.5	35.1	28.0
E	40	30	30	25	350	200	200	200	42.7	30.5	30.5	30.5
F	40	30	30	25	350	200	200	200	42.7	30.5	30.5	30.5
G	55	45	35	35	350	200	200	200	42.7	30.5	30.5	30.5
H					200	200	200	200	30.5	30.5	30.5	30.5
J					550	400	250	175	55.7	46.2	35.2	28.0
K	50	na	35	30	450	na	250	200	49.3	na	35.2	30.5
L	40	30	30	25	350	200	200	200	42.7	30.5	30.5	30.5
M					500	300	200	200	52.6	39.0	30.5	30.5

NOTE: City H SSD from vertical curve sight distance criteria

City J SSD from sight triangle requirement

na - not applicable, city does not have this class

adequacy of horizontal radius standards. Additional assumptions included:

- Cross fall along the outside of the curve was adverse or negative, and
- Maximum safe speed for sight distance could be calculated on the basis of an available horizontal line of sight extending to the edge of the right-of-way line.

Again the assumption of level gradient makes the SSD criterion less rigorous.

To illustrate the method, City K had local residential design speed (30 mph) and radius (430 ft) standards. A 430-ft radius, with the city standard for a 0.0347 ft/ft cross fall, was suitable for 33.6-mph speeds along the outside of the curve with negative cross fall. With the given city street and right-of-way widths, the driver would have a line-of-sight offset (M) of 20 ft. This value of M was measured from the center of the inside lane to the right-of-way line, and it provided a SSD safe for 36.4 mph. The lesser of the two speeds, 33.6 mph, was critical.

$$\Delta = x - x_{MED} = 33.6 - (30 - 6.41) = 10.0 \text{ mph} \quad (6)$$

$$\Delta/\sigma = 10.0/5.0 = 2.0 \quad (7)$$

For $Z = 2.0$ the area under the one-tail normal curve is greater than 0.90. With City K's design speed as the benchmark, the horizontal radius design standard accommodated more than 90 percent of the drivers, for a rank of 1.

Some cities did not report a value of centerline radius to be analyzed. For those cities that did report a centerline radius standard but that did not report design speed, the researchers used the assumed design speeds mentioned above. Table 3 shows the data used for the evaluation.

Grades

When grades are too flat, drainage problems may occur. When grades are too steep, uniform traffic operation is disrupted, heavy

TABLE 3 Evaluation of Horizontal Radius Standards

	CITY DESIGN SPEEDS				CITY MINIMUM RADIUS				MAXIMUM SAFE SPEED, GIVEN CITY RADIUS, CROSSFALL AND BORDER			
CITY	Maj. Art. mph	Min. Art. mph	Col. mph	Loc. Res. mph	Maj. Art. ft.	Min. Art. ft.	Col. ft.	Loc. Res. ft.	Maj. Art. mph	Min. Art. mph	Col. mph	Loc. Res. mph
A												
B												
C					uk	300	100	50	uk	30.4	20.5	14.3
D	60	30	35	30	1412	300	350	100	50.2	30.0	31.5	20.5
E	40	30	30	25								
F	40	30	30	25	400	300	100	100	32.9	30.3	20.7	20.6
G	55	45	35	35								
H					500	250	200	140	35.7	uk	26.1	24.4
J					500	300	100	100	35.3	29.2	20.1	20.1
K	50	na	35	30	1400	na	610	430	50.7	na	38.3	33.6
L	40	30	30	25	400	300	100	100	32.7	29.2	20.1	20.1
M					500	300	100	100	36.0	30.3	uk	20.5

NOTES: City K allows smaller radius when superelevation employed

na - not applicable, city does not have this class

uk - unknown, data missing

vehicles slow too much, and driving on icy streets is complicated. (*1*) Table 4 lists the gradient controls suggested by the Green Book.

The researchers developed a set of desirable and absolute gradients for evaluating the adequacy of each city's gradient standards. They took into consideration that the terrain in most Oklahoma cities is rolling or flat and that land prices are inexpensive in comparison with land prices in other parts of the country. Table 5 lists the recommended maximum and minimum grades in percentages for different functional categories.

The 12 cities furnished standards for grades, but not all cities had values for all categories. Table 6 gives these values.

The researchers evaluated the adequacy of each city's grade standards by comparing them with the desirable and the absolute

TABLE 4 Maximum and Minimum Grades Suggested by the Green Book (*1*)

Functional Class and	Maximum		Minimum	
Green Book reference pages	Desirable %	Absolute %	Desirable %	Absolute %
Arterial 40 mph (p. 525, 235)	7	10	0.50	0.30
Arterial 30 mph (p. 525, 235)	8	11	0.50	0.30
Collector 30 mph (p. 472, 480)	9	12	0.50	0.30
Local res. (p. 435)		15	0.30	0.20

TABLE 5 Recommended Maximum and Minimum Grades

Functional class	Maximum		Minimum	
	Desirable %	Absolute %	Desirable %	Absolute %
Major Arterial	7	10	0.40	0.30
Minor Arterial	8	11	0.40	0.30
Collector	9	12	0.40	0.30
Local Res.	10	15	0.30	0.20

standards. If the city standard maximum grade was equal to or less than the desirable maximum value, the city standard got a rank of 1. If the city grade fell between the desirable and the absolute maximum values, the city standard got a rank of 3. If the city standard grade exceeded the absolute maximum value, it got a rank of 5.

If the city standard minimum grade was equal to or greater than the recommended desirable minimum value, the city standard got a rank of 1. If the grade was between the desirable and the absolute minimum values, the city standard got a rank of 3. If the city standard minimum grade was less than the absolute minimum, it got the lowest rank of 5.

Section Width

The researchers evaluated the section width standards of the cities. The principles considered included the following:

1. It is desirable that the gutters on higher-speed, higher-volume streets be offset from the lane edge, so drivers will have a greater sense of freedom and so depressions in front of inlets will not be in the path of moving vehicles;
2. It is desirable that arterial streets have separate lanes for left-turning vehicles; for the design passenger vehicle to make a me-

TABLE 6 Evaluation of Gradient Standards

City	MAXIMUM GRADE				MINIMUM GRADE			
	Maj. Art.	Min. Art.	Col.	Loc. Res.	Maj. Art.	Min. Art.	Col.	Loc. Res.
A	4.0		4.0	6.0	0.4		0.4	0.4
B	10.0		10.0	10.0	0.5		0.5	0.5
C	6.0		6.0	10.0	0.5		0.5	0.5
D	4.0		6.0	8.0	0.4		0.4	0.4
E	5.0	7.0	10.0	10.0	0.5	0.5	0.5	0.5
F	7.0	7.0	10.0	15.0	0.5	0.5	0.5	0.5
G	5.0	5.0	8.0	8.0	0.5	0.5	0.5	0.5
H	5.0	5.0	8.0	8.0	0.3		0.3	0.3
J	6.0	6.0	6.0	8.0				
K	5.0		8.0	8.0	0.5		0.5	0.5
L	7.0	7.0	10.0	15.0	0.5	0.5	0.5	0.5
M	5.0	7.0					0.4	0.4

NOTE: City H sets 0.5% minimum grade for asphalt streets

TABLE 7 Section Widths for Ranking Criteria

CLASS	MINIMUM FACE-TO-FACE OF CURB WIDTHS TO GET RANK RANK 1	RANK 3	RANK 5
Arterial or Major Arterial	width for 4 through lanes, center median, offset curbs: 48+18+4=70 ft	width for 4 through lanes, center flush median, offset curbs: 44+12+2=58 ft	less than 58 ft.
Minor Arterial	width for 4 through lanes, center flush median, offset curbs: 44+12+2=58 ft	width for 4 through lanes and offset curbs: 44+2=46 ft	less than 46 ft.
Collector	width for 2 through lanes plus parking on both sides: 22+8+8=38 ft	width for 2 through lanes plus parking on one side: 24+8=30 ft	less than 30 ft.
Local	width for one through lane plus parking on both sides: 12+8+8=28 ft.	width for one through lane plus parking on both sides: 10+8+8=26 ft.	less than 26 ft.

dian U-turn from the inside lane to the outside lane, the Green Book (*1*) calls for 24 ft for two lanes plus an 18-ft median;

3. It is desirable that collector streets have at least two moving lanes unimpeded by parked vehicles; and

4. So long as the street's length is limited, it is acceptable for local residential streets to have width for one lane of moving traffic, with parking available on both sides.

Both safety and convenience dictate that urban arterial streets have medians. Getting the left-turning vehicles out of the through lane allows through traffic to maintain speed, allows progressive movement between traffic signals to be maintained, and reduces the potential for rear-end collisions. In a number of passages the Green Book (*1*) suggests that arterial streets have separate lanes for left-turning vehicles in the form of either flush continuous left-turn lane medians or medians with left-turn bays. A 1990 report (*6*) noted that four-lane undivided streets generally have higher accident rates than streets with a median. The report also stated that raised medians were the best technique for preserving the function of through traffic movement and controlling access on an arterial.

The standards for Oklahoma cities were set with consideration of the relatively low price of land. Most parts of Oklahoma were opened to development as late as 1889 to 1900, and a grid of through streets at 1-mi intervals exists in most cities. Present land development is characterized by low densities, and there is usually plenty of open space for wide streets. Table 7 lists the criteria used by the researchers. Table 8 lists the street section widths called for by the city standards. The section standards proposed in various authoritative publications do not fully agree. The criteria used by the researchers are not as rigorous as some; in some categories the researchers' criteria listed narrower lanes, flush me-

TABLE 8 City Street Section Standards

City	Arterial or Major Arterial ft	Minor Arterial ft	Collector ft	Local ft
A	64	44	40	27
B	47	47	37	26
C	?	44	32	26
D	87	50	36	26
E	48	44	32	26
F	50	50	32	26
G	52	52	32	26
H	50	na	34	26
J	48	48	32	26
K	48	na	32	26
L	50	50	32	26
M	62	48	32	28

dians, and curb offsets in comparison with the recommendations of many others.

Intersection Radius

The curbs of two intersecting streets are joined not at a right angle but with a short curve. If this short curve has an unnecessarily large intersection radius, the distance required for the pedestrian crossing movement is lengthened (*4*). A larger radius may also increase the frequency of "rolling stops" or encourage higher turning speeds (*4*). On the other hand, inadequate radii result in vehicles bumping the curb. From the driver's point of view, the intersection radius needs to be large enough for most vehicles to turn without having to turn at a crawl speed or without bumping into the curb while turning (*7*).

The researchers evaluated the adequacy of city radius standards for arterial-arterial, arterial-collector, and collector-collector intersections. The adequacy of a radius is a function of the width of the lane turned from, the width of the lane turned into, and the design vehicle. The researchers used the design vehicles listed in Table 9 for evaluating adequacy.

The researchers used city street lane widths in combination with the standard values for intersection radius to make scaled drawings of typical intersections. Each intersection radius was evaluated by overlaying vehicle turning templates onto the scaled intersection drawings. The design vehicle began the right rurn entirely within the right lane and completed the turn without striking the curb. A ranking was then given on the following basis:

Rank 1, vehicle made a 90-degree turn without entering the lane for opposing flow on the street turned into (i.e., did not cross the centerline);

Rank 3, vehicle made the turn but jutted out less than 1 ft into the oncoming lane; or

Rank 5, vehicle made the 90-degree turn but jutted out more than 1 ft into the oncoming lane.

For example, a city might specify a 50-ft-wide arterial section with four lanes and a 30 ft radius. If the WB-50 vehicle template made a 90-degree turn at the intersection of two arterials, from a 12-ft lane into a street half-width of 25 ft without crossing the centerline, then the rank given was 1.

RESULTS

The researchers evaluated the adequacy of city design standards intended for new developments or streets. Table 10 gives the results in the form of rankings of the quality of various city geometric design standards. The gaps in Table 10 reflect the absence of a functional class in a particular city, the absence of a standard for a particular design issue, or a missing standard for a particular functional class.

It is noteworthy that a number of cities had incongruous standards, in the sense that the speed for which the sight distance was suitable differed greatly from the city's design speed. Some of the city design speeds are rather high and probably could be decreased. On the other hand the 30-mph design speed that a number of the cities had adopted for minor arterials is probably too low for Oklahoma conditions. The present study indicated that despite the availability of published state-of-the-practice design criteria, some of the fundamental geometric standards at the local level were substandard. Standards for major arterials were most in need of improvement.

In general, the cities had SSD standards that were adequate for the design speed. However, half the cities needed to revise their SSD standards for their major arterials. Standards for SSD should consider the effects of downgrades on stopping distances.

The city standards for horizontal centerline radii did not accommodate the drivers' needs (i.e., the design speed) in many instances. Only City K had consistently adequate minimum radius standards. The assumption that the line of sight was clear up to the right-of-way line may have been overly generous; it may be that city standards for horizontal curvature are actually worse than the evaluation indicated.

In almost every instance city standards for gradient met or exceeded those recommended by the Green Book (*1*). The cities' gradient standards scored better than the other design issues evaluated.

In most instances, the cities' standards for street section width were marginally adequate (Rank 3), but could improve. Arterials were an exception; most cities needed to call for wider major arterial sections.

The cities had adequate to good intersection turning operations, as measured by the lane widths and radii. There were data to evaluate only 8 of the 12 cities' intersection standards.

The researchers had to interpret some of the standards and could have made a mistake in so doing. In a few cases the various design documents for a particular city did not agree with each other.

Some of the cities contacted did not have standards for such fundamental design criteria as minimum SSDs, and during the interviews some city engineers did not understand such concepts as a "design vehicle" for geometric layout controls. A city staffer from one of the cities that did not respond to the survey said that that city had no geometric design standards.

The widespread horizontal radius deficiencies cause one to wonder whether some local staff appreciate fundamental design issues. The need to remove left-turning vehicles from the traffic stream to preserve the functional demands of major arterial traffic often seemed to be ignored.

TABLE 9 Recommended Intersection Design Vehicles

	Collector	Arterial
Collector	Bus	WB-50
Arterial	WB-50	WB-50

RECOMMENDATIONS

Many of the suggested design criteria contained in the Green Book (*1*) and other recognized publications are based on the scientific studies of the limitations and capabilities of drivers, vehicles, and roadways. It is doubtful that the engineer at the local level can rationally justify design standards that vary greatly from those suggested by the experts. If the recommended practices constitute a valid yardstick, then some city street design standards do

TABLE 10 Ranking of City Standards

	A	B	C	D	E	F	G	H	J	K	L	M
ARTERIALS AND												
MAJOR ARTERIALS												
SSD	2			5	1	1	5	5	1	2	1	1
Centerline radius			uk	5		5		4	4	1	5	4
Grade - maximum	1	3	1	1	1	1	1	1	1	1	1	1
Grade - minimum	1	1	1	1	1	1	1	3	uk	1	1	uk
Section width	3	5		1	5	5	5	5	5	5	5	3
MINOR ARTERIALS												
SSD				1	1	1	5	4	1		1	1
Centerline radius			4	1		1		uk	5		2	4
Grade - maximum					1	1	1	1	1		1	1
Grade - minimum					1	1	1	uk	uk		1	1
Section width	5	3	5	3	5	3	3		3	na	3	3
COLLECTORS												
SSD	1			1	1	1	4	1	1	1	1	1
Centerline radius			5	4		5		4	5	1	5	uk
Grade - maximum	1	3	1	1	3	3	1	1	1	1	3	uk
Grade - minimum	1	1	1	1	1	1	1	3	uk	1	1	1
Section width	1	3	3	3	3	3	3	3	3	3	3	3
RESIDENTIAL LOCALS												
SSD	1			2	1	1	4	1	1	1	1	1
Centerline radius			5	5		4		2	4	1	4	4
Grade - maximum	1	1	1	1	1	3	1	1	1	1	3	uk
Grade - minimum	1	1	1	1	1	1	1	1	uk	1	1	1
Section width	3	3	3	3	3	3	3	3	3	3	3	3
INTERSECTION RADIUS												
Arterial w/ arterial	1		3	1		3	3	1		1	3	
Arterial w/ collector	3		5	1		3	3	1		1	3	
Collector w/ collector	1		1	1		1	1	1		1	1	

not measure up. Although the standards from only one state were studied, it would be odd if the problem of substandard standards were confined to one state. It is more likely that local design standards in other states also need improvement.

If local design standards do in fact need improvement, there are a number of possible ways to proceed, including

1. Do nothing,
2. Conduct education and extension programs, or
3. Invoke federal or state involvement and regulation.

To choose the "do nothing" option, it appears that one would have to conclude that no significant problems were being caused by cities having standards below the state of the practice. A rebuttal to this position is that, to accept less than the state of the practice, one must be unaware of the research-based principles on which state of the practice is founded.

A common prescription in U.S. society for curing a deficiency or problem is more education. One possible remedy for inadequate city design standards is more education for city officials. A rebuttal to this argument is that the inhibiting factors mentioned earlier plus time constraints sometimes stifle change at the local level. Also the record shows that local government officials are not always receptive to or able to implement improved and progressive engineering practices. If city officials were overwhelmingly proactive, how can one explain the need to have outside pressures (e.g., lawsuits and legislative mandates) to make cities take action? A case in point was that cities did not improve wastewater treatment systems until they were forced to do so by the federal government.

In recent years another common approach to addressing problems has been federal or state intervention. A disadvantage to federal or state standards would be the creation of more regulations and bureaucracy; some will oppose intervention on the basis of their philosophical biases, regardless of the facts or needs. The principle of the federal government setting minimum environmental (wastewater, storm water, etc.) requirements that local governments must meet could be applied to transportation; the federal or state governments may need to mandate local government street standards for a few design topics. The following factors suggest this need:

1. Local governments may lack the funds to hire personnel with the expertise to establish a comprehensive, up-to-date set of street design standards;
2. Some local government officials and staff do not always appreciate the need to implement progressive geometric design controls; and
3. Local political environments may not be conducive to establishing adequate, modern street design standards, especially when such standards would impose more stringent requirements than those currently in effect.

A set of uniform minimum standards would benefit the engineering design community if the result were a reduction in differences among city standards, which would in turn reduce the number of different practices with which engineers would have to cope. More important, the general public would benefit, because a higher level of safety and convenience would be built into the street network if inadequate standards were overridden by mandated practices. Perhaps future exposure to tort lawsuits would be reduced, saving the taxpayers' money.

CLOSING

Transportation is not a local issue now any more than the wastewater issue was in recent years. In the typical U.S. metropolitan area one city merges into the next. Travel and commerce move from city to city and state to state; travel does not recognize the city limit. Like wastewater traffic congestion and accidents affect people downstream of a city. If the engineering design criteria recommended by the experts are reasonable and if it is important that the public be afforded a minimum level of quality, then engineering leaders need to devise methods that actually bring quality to the public. If the current method if ensuring quality is not working, another method should be considered.

An analysis of one state's data does not prove a widespread need, but it suggests further investigation. Section 1049 of the 1991 federal transportation bill (Intermodal Surface Transportation Efficiency Act) mandates a review of state standards relating to design. Although research often emphasizes state highway issues, local government needs should not be overlooked: there should be more evaluations of the quality or adequacy of city design practices. It may be that city design standards are adequate in some regions but not in others. There will be legitimate reasons for differences among cities' standards, but at some point differences can cross the line into the realm of inferior practice. If standards are mandated, it will take effort to build standards that control practices that do not measure up to the criteria that are based on engineering science without interfering with legitimate differences. Mandating a few fundamental city street design standards should not have a significant impact on the affairs of those cities that have adequate transportation design standards. Only those with inadequate standards, the ones not delivering a certain level of quality to the public, would feel a significant impact.

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Design Consistency and Driver Error

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Geometric design consistency appears to be a major factor affecting accident rates on rural highways, yet little assistance that enables engineers to design roadways consistent with driver expectations is available. AASHTO instead focuses primarily on individual elements, basing guidelines on functional classification, volume, and design speed. The methods that have been presented in the literature for quantitatively assessing design consistency are focused in two primary directions, speed consistency and driver workload. Speed consistency consists of analyzing predicted speeds on a highway and striving to keep those speeds within a narrow range. Several major research studies have provided methodologies for deriving and analyzing predicted speeds. Workload consistency for geometric design, however, has been the focus of only one major research study, receiving an examination by Messer et al. in 1981. In their study they developed procedures that assign subjective workload ratings for features along the roadway, depending on the type and severity of features, the sequence of features, and the proximity to other features. Some 19 rural two-lane highways in Texas were analyzed to derive relationships between the workload ratings provided by the procedure of Messer et al. and the accident records on those roadways. It was concluded that roadway sections with either high workload magnitudes or large positive changes in workload were associated with high accident rates when compared with accident rates on other sections on the study roadways. A final conclusion was that the driver workload procedure of Messer et al. represents a viable tool for use in the examination of design consistency.

Roadway designers are faced with many choices in the design or rehabilitation of a roadway. The designer must meet or exceed the requirements placed by engineering guidelines and standards, choosing, in some cases, which requirements will be met. The designer must then request design exceptions for those requirements not met. One of the requirements placed on the designer is to meet driver expectancies (*1*). Given the vague guidance provided in this area, most of the designer's attention is usually directed toward the clear-cut requirements for discrete elements of the design, neglecting an overall examination of the driving environment. When today's designer attempts to reconstruct segments of old routes as needs dictate and money becomes available, attention must be placed on the issue of driver expectancy so that inconsistencies are not built into the highway system.

DESIGN CONSISTENCY

Highway designers are vitally concerned with building the most efficient, most cost effective, and safest highways possible. To increase the safety of a highway, however, the engineer must know which portions of the roadway merit improvement. The necessity for accident prediction by the transportation engineers was pointed out by Dart and Mann when they stated, "Unless accidents can be predicted above the level of chance, the processes that cause accidents cannot be understood with any degree of confidence" (*2*). Accidents do not occur in the same locations every year, though, and patterns in accident occurrence are difficult to understand and quantify, confounding the engineer's attempts to provide a safer driving environment.

Little need would exist for accident study and analysis if drivers could readily assess the risks that they encounter as they drive. A British study by Watts and Quimby (*3*), however, compared the risks encountered with the drivers' perceptions of those risks and found that wide discrepancies existed between the objective and subjective risk levels. This discrepancy was later confirmed by Philput (*4*) using U.S. drivers. Because drivers do not appear to be capable of accurately assessing the risks they encounter by driving along a roadway, they are not able to adequately modify their driving behaviors accordingly.

Driver Expectancy

Expectancy, in general, can be stated to represent a set of possible probabilities regarding a given situation (*5*). Those probabilities are subjective and based on learned and experienced events. Expectancy is a known determinant of reaction time, signal detection, and vigilance. Because the driving task involves all these factors, attention must be placed on the driver's expectancies. An operational definition of expectancy has been given by Ellis (*6*):

> Driver expectancy relates to the observable, measurable features of the driving environment which:
> (1) Increase a driver's *readiness* to perform a driving task in a particular manner, and
> (2) Cause the driver to *continue* in the task until it is completed or interrupted.

This definition attempts to narrow expectancy from the general view of the psychology profession to the viewpoint of the transportation engineer.

Speed Differential

A driver's expectancy of a specific situation is formed by his or her experience, both long term and short term (*6*). Long-term expectancy has been termed *a priori expectancy*, and short-term expectancy has been termed *ad hoc expectancy* (*7*). Expectancy influences many of the decisions encountered in the driving task. Some researchers have examined design consistency in an attempt to provide a consistent roadway design. One method of examining the consistency of a roadway design is to check various surrogate measures. Although several surrogates are available (*8*), one mea-

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sure that appears useful is speed differential along the roadway. This particular measure of operation has the advantage of being both easily measured and easily simulated, permitting the simulation of roadways under study and the measurement of speeds along existing roadways for research purposes. Several methodologies have been developed for use in analyzing speed differential. These techniques generally focus on reducing or limiting the severity of speed differential along the roadway.

Lamm et al.

Research has focused on ways of providing a consistent speed environment that conforms to driver expectancies and does not require abrupt changes in operating speed to maintain control of the vehicle. Several different design consistency procedures have been presented in the literature. One of the simpler models was presented by Lamm et al. (*9*). This procedure concentrates on operating speed changes induced by horizontal curvature and tangent length, as well as examination of the change in degree of curvature of horizontal curves along the roadway. The strategy focuses on achieving a consistent horizontal alignment by minimizing abrupt changes in operating speed, while keeping the change in degree of curvature to a minimum.

Leisch and Leisch

A procedure introduced by Leisch and Leisch (*10*) includes the influence of both horizontal curvature and vertical grade. Variations in automobile speeds of more than 10 mph, reductions in design speed by more than 10 mph, and differences in speed between trucks and automobiles of more than 10 mph are to be avoided. The objective of the procedure is to enable the designer to detect areas of the highway alignment that violate these recommendations. Truck speeds are predicted from tabular values presented in AASHO's 1965 *A Policy on Geometric Design of Rural Highways* (*11*), whereas the speeds of automobiles are determined through the use of equations derived from driver characteristics. Leisch and Leisch's (*10*) approach to design consistency considers many operational characteristics of the roadway-driver system.

Switzerland

Switzerland (*12*) uses both a design speed and a project speed for arriving at proposed alignments for highways. The design speed is used in a manner similar to that in which it is used in the United States' AASHTO guidelines (*1*). The design speed provides a minimum design value for various roadway features (i.e., sight distance, horizontal curvature, etc.), whereas the project speed is the "maximum speed expected in a certain roadway section and serves as a test speed to assess adequate sight distances, adequate radii of crest or sag vertical curves. . . ." Switzerland uses a speed model to examine the horizontal roadway alignment, predicting project speeds throughout the alignment. By examining changes in that project speed, abrupt changes in speed as well as speed transitions along the roadway may be detected.

Germany

Germany also uses both a design speed and an operating speed as aids in roadway design (*12*). Operating speed as defined in Germany corresponds to the 85th percentile speed on a facility. An acceptable alignment would have a predicted operating speed that did not exceed the design speed by more than 20 km/hr (12 mph). German designers also use the effects of alignment on speed to deliberately provide a speed transition when passing from high-speed rural areas to low-speed populated areas, introducing curvature that might otherwise be unnecessary. Speed transitions are controlled by examining a "curvature change rate" to ensure that transitions are gradual and safe between adjacent roadway sections. Other checks on horizontal alignment include controls on successive curves, tangent lengths, and the number and severity of curves along stretches of roadway.

Australia

Design consistency research by McLean (*13*) has focused on the differences between design speeds and desired speeds. Desired speed has been defined as the 85th percentile speed measured on tangent sections of roadway within a particular roadway section. For high-speed alignments Australian practice is to continue to provide the conservative design features that have been provided previously, since this practice has proven to be consistent with driver expectations and practices. For low-speed alignments, however, design speed is made to match the 85th percentile speed. Following the preliminary selection of horizontal curve radii, projected speeds are estimated for the curves. Those speeds are then used to specify other parameters for the design.

Summary

All of the methodologies discussed so far have concentrated on treating roadways so that *observed* driver responses (i.e., driver speed changes) conform to specific ranges that have been determined acceptable. A premise implicit in those methodologies, however, is that drivers are able to observe and analyze the roadway in such a way as to arrive at an appropriate speed. If drivers tend to drive too fast on severe curves that follow tangents, however, observation of the speed changes between the two features will result in an underestimation of the design consistency. Watts and Quimby (*3*) established that drivers do not always recognize the risk potential of roadway features; it appears reasonable to question procedures that rely strictly on the results of driver-controlled speed changes. Further examination of consistency and expectancy leads to the workload concept in an attempt to address more directly the influence of the roadway on the driver.

Workload

The driving task imposes work on the driver; this work varies greatly in task difficulty and task frequency. The level of this workload and its effects on driver performance would seem to be greatly affected by driver expectations and driver capabilities. Roadways with inconsistencies in their designs would be expected to violate driver expectancies and impose higher workloads on drivers. An understanding of the basis of workload and its impacts on performance is desirable to analyze design consistency.

Definition of Workload

Workload has been defined by Senders (*14*) as "a measure of the 'effort' expended by a human operator while performing a task, independently of the performance of the task itself." Another definition of workload was given by Knowles (*15*) as consisting of the answer to two questions: "How much attention is required?" and "How well will the operator be able to perform additional tasks?" The definition presented by Knowles seems very appropriate to the driving environment, since it consists of many overlapping tasks, each requiring a portion of the driver's attention. A method of examining the workload demands placed on the driver would appear to be a way of directly arriving at the capabilities of the driver as he or she negotiates a given roadway.

Messer Driver Workload Procedure

A method of evaluating driver workload was presented by Messer (*16*) and Messer et al. (*17*). By gathering empirical evidence regarding driver expectations of roadway features and relating violations of those expectancies to workload, a model was formed. The model is based on the presumption that the roadway itself provides most of the information that the driver uses to control his or her vehicle; hence the roadway imposes a workload on the driver. This workload is higher during encounters with complex geometric features and can be dramatically higher when drivers are surprised by encounters with unexpected or unusual combinations and sequences of geometric features.

The Messer driver workload procedure quantifies design consistency by computing a value for driver workload. The technique relies on a set of assigned ratings for various roadway elements. The following roadway features receive ratings (listed in order of severity): bridges, divided highway transitions, lane drops, intersections, railroad grade crossings, shoulder-width changes, alignment, lane-width reductions, and the presence of crossroad overpasses. The ratings, based on the type and severity of design element, are then modified in accordance with their locations. Influencing factors include sight distance to the element, similarity to previous elements, workload of previous segments, and percentage of drivers estimated to be familiar users of the facility.

The workload along the roadway is estimated by using an equation that defines a subjective level of consistency (LOC) in terms related to driver workload. The methodology is applicable to two- or four-lane highways in flat or rolling terrain and may be used to examine existing or proposed highways. The equation used for calculating the driver workload is defined as:

$$WL_n = UESR_f + CWL_l \quad (1)$$

where

WL_n = workload,
U = driver familiarity factor,
E = feature expectancy factor,
S = sight distance factor,
R_f = basic workload potential rating,
C = carryover factor, and
WL_l = workload of the previous feature.

The range of each of these terms was defined by the collection of empirical evidence and the collective experience of a group of experts (*16,17*). The first addend represents the workload associated with the feature in question, whereas the second addend represents the residual effects of the workload of the previous feature encountered by the driver.

Sample Application of Various Consistency Measures

Several different consistency and workload measures have been presented in the literature. Although a comparison of procedures on any one given roadway does not completely address differences and similarities between the various procedures, such a comparison was of interest. Lamm et al. (*18*) presented a hypothetical alignment (Figure 1) for which three different speed consistency analyses have been performed. McLean (*13*) used the same hypothetical alignment to contrast an Australian methodology with the three speed consistency analyses presented by Lamm et al. (*18*). A fifth measure of consistency, based on workload (*16,17*), has been added through the application of the Messer procedure (*16,17*). The results are shown in Table 1. It is highly significant that all four checks of operating speed rejected the same two curves (AB and EF). This agreement shows a very

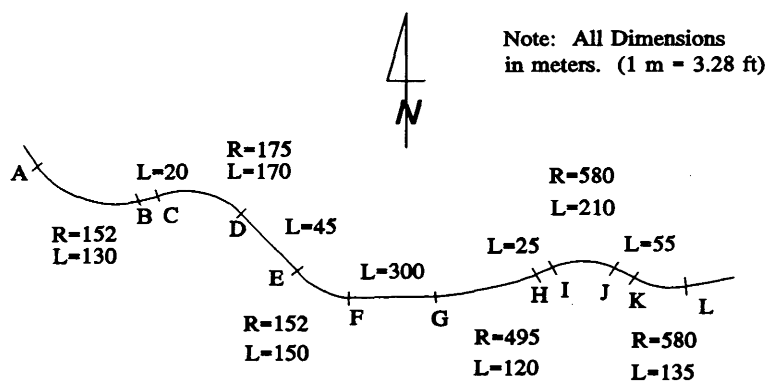

FIGURE 1 Hypothetical alignment.

TABLE 1 Operating Speed Predictions (*15,23*) (km/hr) (NAASRA is National Association of Australian State Road Authorities)

Method	Curve					
	AB	CD	EF	GH	IJ	KL
Leisch W-E	60*	63	60	71	76	85
Leisch E-W	60	63	60*	93	97	97
Swiss	69*	69	69*	100	100	100
German	70*	70	70*	86	86	86
NAASRA W-E	85+	77	76	91	100	100
NAASRA E-W	76	77	82*	99	100	100
†Messer W-E	F♦	F♦	F♦	B	B	B
†Messer E-W	E	F♦	F♦	E	D	B

* Unacceptable because of speed consistency criterion.
\+ Unacceptable because of side friction factor criterion.
♦ Unacceptable because of high workload.
† Added by this author.

promising similarity of results, suggesting a convergence of research and methodology.

Although it does appear to be a promising technique for evaluating geometric roadway alignment, limitations of the speed profile methodology of examining design consistency are numerous. All the procedures examined ignore the influence of many design elements, such as intersections, narrow bridges, lane width changes, changes in the number of lanes, and so on (*19*). All of these features violate drivers' expectancies (*16,17*), yet their influences on the geometric design of the roadway are neglected if one examines only the speed profiles along the roadway.

A fifth measure of consistency, that provided by the application of the Messer driver workload procedure (*16,17*), is different in implication when applied to the hypothetical alignment of Lamm et al. (*18*) (added by the author). The results from the Messer procedure are reported in a range extending from A (no problem expected) to F (big problem possible). In addition to curves AB and EF in Figure 1, the Messer procedure predicts that problems may be predicted for curve CD for the west to east (W–E) direction of travel, and curves CD, GH, and IJ could present problems as well (although to a lesser extent). The application of the Messer procedure was completed by using an assumed 140-ft right-of-way (ROW) in the absence of further information; a further assumption was made, that is, that sight was restricted at the limits of that ROW; and a final assumption was made to use the speeds predicted by Leisch and Leisch (*10*) to represent the 85th percentile speeds on the alignment.

Clearly, significant differences are evident in the application of the various procedures outlined above. Although further information regarding the hypothetical alignment (e.g., providing a vertical alignment and showing structures) would provide a better comparison of the methodologies, this simple example does provide a basis for comparison. The Messer procedure (*16,17*) is highly sensitive to severe horizontal curvature; this sensitivity generally accounts for the differences in results. The speed profile procedures assume that once a driver has slowed for one curve, another similar curve is of little consequence; Messer, on the other hand, using workload, assumes that combining high-workload features in close proximity results in a higher workload for the second and following features. The different assumptions made by the respective researchers hence affect the results.

STUDY RELATING DRIVER WORKLOAD AND ACCIDENT EXPERIENCE

To validate a procedure that purports to enable the designer to improve safety, an examination of accident experience on actual roadways is necessary. This section reports the results of a study that examined one class of two-lane roadway in Texas (*20*). The hypothesis investigated in that research was that accident rates would be highest in areas associated with either very high workloads or extremely low workloads. A corollary to this hypothesis was that sudden increases or decreases in workload would also be associated with increased accident risk. As theorized in the Yerkes-Dodson Law (shown in Figure 2), performance level is low when arousal level is low. Performance gradually improves in quality as arousal increases until an inflection point is reached; after that point is passed and arousal continues to increase, performance declines (*21,22*). Since error rates generally increase as performance decreases, it was expected that accident rates would be highest in those areas where workloads were minimized or maximized. The general shape of the hypothesized relationship between workload and accident rate is illustrated in Figure 3.

Application of Messer Driver Workload Procedure

The Messer driver workload procedure (*16,17*) was applied to selected portions of 19 different farm-to-market roadways in Texas. Information regarding the functional and geometric characteristics of the study roadways was obtained from Texas Department of Transportation (TxDOT) personnel. Roadways selected for study were two-lane rural highways. The study roadways were functionally classified as rural collectors, were in rolling terrain, and had reasonably consistent traffic volumes. Speed limits on the roadways were 55 mph, with lower advisory speed limits on some sections.

Driver workloads were calculated for the study roadways by the Messer driver workload procedure. Geometric features such as vertical and horizontal curvatures, presence of intersections, and so on, were determined, and then various charts were consulted to determine feature ratings. Once the individual feature ratings were determined, various modifying factors such as sight distance and expectation (or familiarity) were calculated. Each feature along the roadway segment was then assigned a workload (*16,17*).

Accident History

The validation of the Messer procedure with regard to accident history required that accident records be obtained for each of the

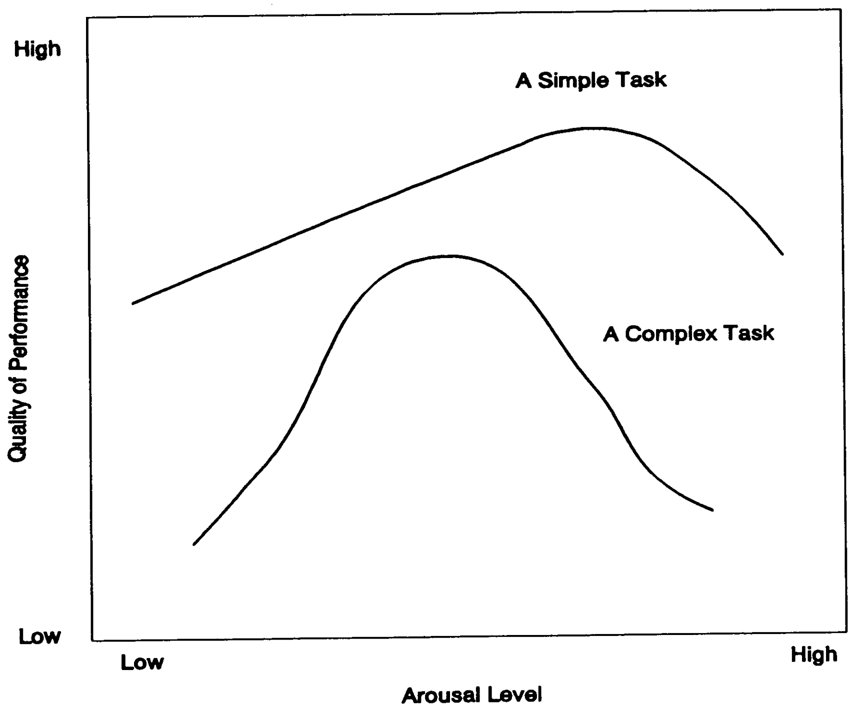

FIGURE 2 Yerkes-Dodson Law.

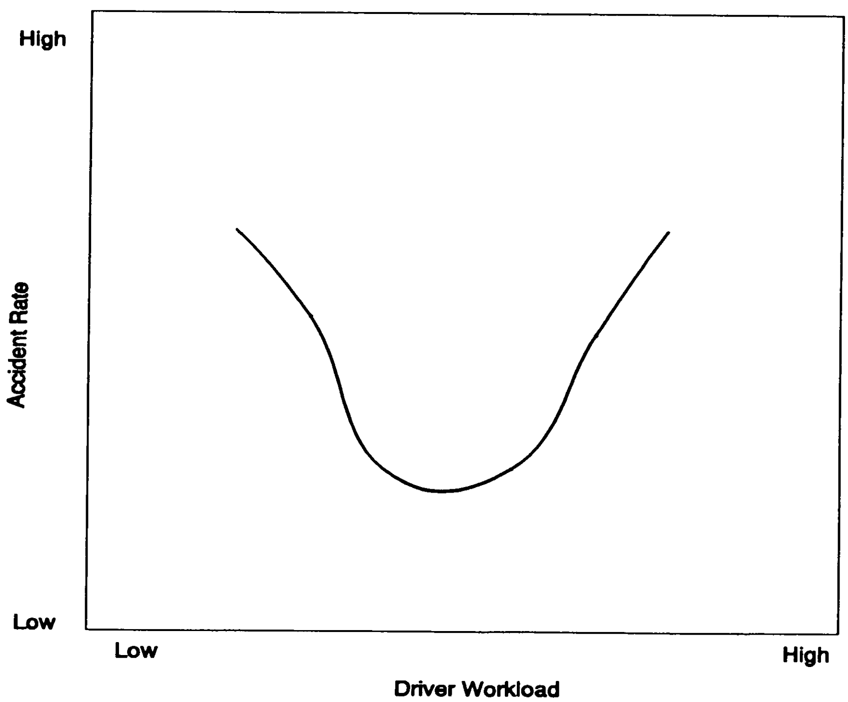

FIGURE 3 Hypothesized relationship between workload and accident rate.

roadway segments in the study. Since the Messer procedure is applied to travel directions of a highway independently, it was necessary to separate accidents by direction. Accident records for each roadway were obtained from TxDOT. The roadways that were included from Glascock's study (*20*) included accidents from January 1987 to June 1990 (3 years 6 months); the accidents listed for roadways added for the present study included accidents from January 1988 to May 1991 (3 years 5 months).

The length of the time period chosen, approximately 3.5 years, represented a compromise between instability in accident occurrence and instability in site characteristics. Accidents are random in nature, and accident occurrence fluctuates accordingly. Yearly numbers of accidents fluctuate, so it is desirable to use as many years of data as possible; however, site characteristics are changeable and dictate that short time periods be used to ensure that site conditions remain reasonably constant. The period of 3.5 years represented a compromise between these two conflicting requirements and was used in the study (*23*).

Data Analysis

An evaluation associating driver workload with accident rates was performed. The evaluation related driver workloads associated with individual portions of roadway with the accident rates on those portions of roadway. The microscopic evaluation was accomplished by using two individual variables. In an attempt to determine the influence of a priori expectancy, a grouping was made of those segments with workloads within the six levels of consistency presented by Messer. Ad hoc expectancy was examined through the determination of the yaw of the workload on individual features. *Yaw* was defined as the difference between the moving average workload and a specific feature's workload. In this way the yaw provided a measure of the change in workload on a microscopic level.

The analysis examined accident rates for roadway features after first sorting them for workload rating and workload yaw. Yaw was calculated according to the following equation:

$$\text{Yaw} = (\text{workload rating of feature}) - (\text{moving average workload}) \qquad (2)$$

The extent of roadway length considered in the determination of the moving average was 1,000 ft. This length was determined after consideration of research by Lamm et al. (*9*) as well as Matthews and Barnes (*24*). The use by Lamm et al. (*9*) of an independent or nonindependent tangent length when the tangent length between successive curves influenced the level of consistency of the following curves led to an examination of the lengths used in their proposed methodology. The lengths at which the 85th percentile speed (V_{85}) in the tangent could be expected to be 58 mph (or maximum) ranged from 1,100 to 475 ft, depending on the V_{85} in the curve. In Matthews and Barnes' research (*24*), accident risks were determined to be four to seven times as high for curves located immediately after 300-m (984-ft) or longer tangents when compared with the accident risks on standard curves. Extending their research to the workload concept, it was decided to examine each feature's associated workload in relation to the workload in the previous 1,000 ft. Both the workload ratings and the yaws were grouped into ranges, and the numbers and cumulative lengths of features with those characteristics were determined.

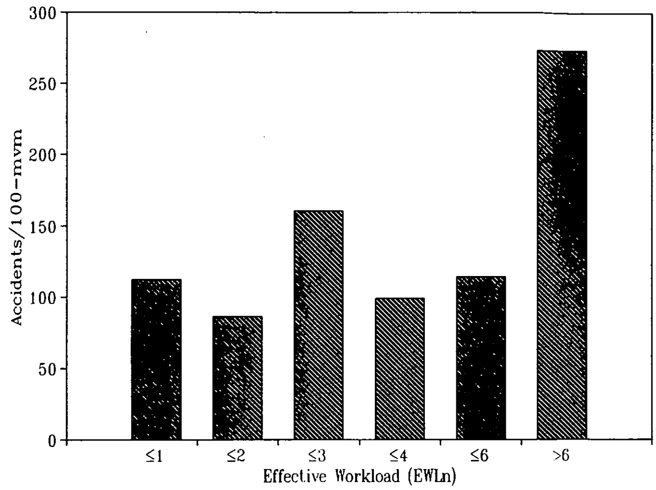

FIGURE 4 Evaluation of effective workload.

The number of accidents in those grouped features was then determined.

The roadway features were grouped in two different schemes. The first scheme grouped features according to Messer's proposed LOC, which ranged from A to F. The number of accidents per 10^8 vehicle mi was then calculated for the cell groupings. The results are given in Figure 4. As may be seen from the graph in Figure 3, accident rates dramatically increased for those features with effective workloads of greater than 6 (LOC F). This finding is consistent with Messer's projected "big problems possible" for this LOC (*16,17*). In a second grouping scheme, roadway segments belonging to various categories of yaw were grouped. As shown in Figure 5 the accident rate was much higher for the segments with yaws of greater than or equal to 4.

Those roadway segments that had high effective workloads and those that had effective workloads much higher than the moving average had much higher accident rates than those segments that had lower effective workloads and those segments that had low yaw values. This finding was expected; an elevated accident rate

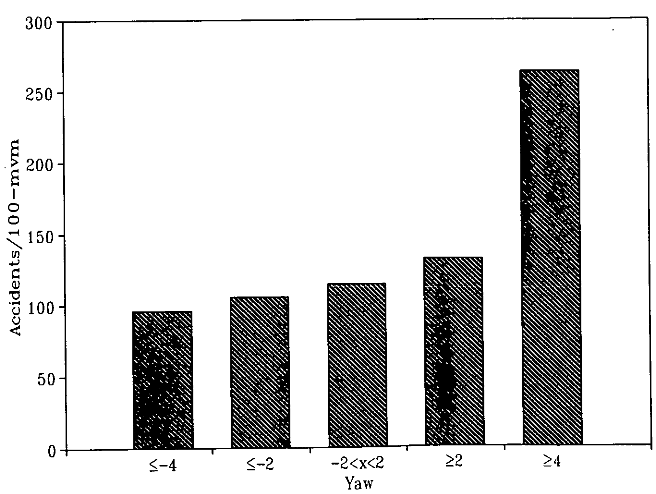

FIGURE 5 Evaluation of yaw.

for areas with extremely low workloads and yaws was not found, however. A possible cause for this might have been that the driver workloads on the study roadways might not have been low enough in absolute terms, since roadways were chosen by criteria that included the requirement that all of the roadways pass through rolling terrain. In general, the study roadways had a larger amount of topographical relief and roadway curvature than most roadways, which produced higher driver workloads than are found on most roadways.

CONCLUSIONS AND RECOMMENDATIONS

The 19 highway segments that were studied all shared some basic characteristics: they all functioned as rural collector highways, and they were all defined as passing through rolling terrain. In addition they were generally designed and constructed in the late 1940s to the late 1950s. Although some were reconstructed at later dates, generally most of the alignments remained unchanged from the original time of construction. The roadways are presumed to have met the standards in place at the time of construction, although when examined according to today's standards and guidelines the roadways appeared to be in need of improvement. Vertical and horizontal curvatures were severe, sight distance was limited, shoulders were generally lacking, and bridges sometimes lacked approach guardrail or adequate bridge rail. Despite these and other problems, the roadways remain in active use and will presumably remain in active use for some time. An evaluation of accident experience on the roadways was undertaken in an attempt to determine if the various measures of workload might be related to that experience.

Conclusions

1. The microscopic evaluation of the study roadways showed that large changes in workload over a short distance were strongly associated with high accident rates. When feature workloads were compared with the average workload in the previous 1,000 ft, it was found that roadway segments exhibiting a large positive change in workload experienced a greatly increased accident rate when compared with those on other segments of the study roadways. This finding would seem to indicate that when ad hoc driver expectancies are not met, accident risk increases.
2. The microscopic evaluation of the study roadways showed that segments associated with high workloads (LOC F) were also associated with high accident rates. The accident rates for those segments were much higher than those for the other roadway segments. Although conclusive statistical evidence has not been provided, the available information seems to support Messer's contention that features with high workloads can be expected to have "big problems." This finding would seem to indicate that when a priori expectancies are not met, accident risk increases.
3. The Messer driver workload procedure (*16,17*) was found to be a practical means of assessing design consistency and driver workload. The application of the procedure and the relationship between procedure results and accident history indicate that the procedure is a demonstrated, viable means of analyzing geometric design consistency and driver workload in terms of accident risk.

Recommendations for Future Research

1. A logical next step in the analysis of the Messer procedure would be to couple a study of driver workload with a study of speed variations along a series of roadways. In this way levels of workload could be more precisely calculated (since the V_{85} is one input to the Messer procedure), and the findings could be compared with those recommendations made by various speed consistency procedures (*9,10,13,16,17*).
2. Another area in need of research is the further refinement and extension of the Messer procedure through reexamination of the levels of workload obtained for various roadway features. One way that this objective could be accomplished would be through the use of the occluded vision device currently being tested at the Texas Transportation Institute. The device lets drivers control explicitly the amount of information that they receive through regulation of their sight. Drivers determine the amount of vision time that they need to operate a vehicle as they drive; presumably drivers increase the amount of time they have clear vision during those times when high-workload areas are being traversed. By monitoring error rates, it is possible to screen out those drivers who are overly brave or optimistic about being able to drive a feature. This screening ability could be one mechanism that could further validate the Messer procedure as well as extend the guidelines provided by Messer.
3. Another area that appears to be in need of further research is the concept implicit in the yaw variable used in the examination of driver workload. Further study and analysis of the effects of large abrupt increases in workload seem justified given the relationship between yaw and accident risk revealed in the present study.
4. One last area of research that could prove to be helpful would be to validate the Messer procedure through the study of higher-class roadways, including four-lane divided highways. Although not substantiated by the present research, elevated accident risk is expected on segments of roadway associated with extremely low workloads. It seems reasonable to assume that traffic volume provides a significant part of the workload that a driver experiences; high-standard roadways with low traffic volumes might well experience high accident risks. Texas (and other states) is in the process of forming a trunk system consisting of four-lane divided highways with high standards; many of those roadways will have very low traffic volumes. Through the study of roadways with these characteristics, it would be possible to predict whether these roadways will experience increased accident rates when compared with those on other, similar facilities.

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Design Exceptions: Legal Aspects

STEPHEN F. J. MARTIN

The legal implications of the design exception process in the context of tort liability and tort litigation are examined. Only six states currently report that their highway or transportation departments are immune from suit by motorists injured as a result of alleged defects of the highway. In those states susceptible to suit, the alleged defect may well be a design defect. In such cases the application of design standards and adequate justification of any deviation from the applicable standard become paramount in the defense of the lawsuit. A brief introduction to the notion of sovereign immunity of state governments and some of the important concepts of tort law are offered. Within this context the relationship of design standards to legal notice of highway hazards is explored, and the significance of departure from those standards is explained. Recommendations for the designer are provided. The author's experience, since 1987, as a member of the legal staff of the Pennsylvania Department of Transportation involved in the risk management training and education of Pennsylvania Department of Transportation employees is used as the basis. The recommendations are based on those made to Pennsylvania highway designers and design liaison engineers.

It is not too broad a generalization to state that the law of tort liability or negligence in most states imposes a legal duty on state and municipal governments to provide safe highways and streets for the citizens who travel them. (Tort liability or negligence law in the United States is largely a matter of state, not federal, law and is therefore not uniform from state to state.) In many states that duty encompasses the construction, maintenance, and signing of roads. In some states it also includes the *design* of the streets and highways under the jurisdiction of the state government. It is the purpose of this paper to discuss the notion of legal duty as it relates to the design of highways and, more specifically, to discuss the legal implications of the request for and approval of design exceptions.

HISTORICAL BACKGROUND

The law of tort liability or negligence in the United States developed as a matter of common law from legal concepts and principles brought with the English colonists who settled in North America. *Tort* is the legal term used to describe an actionable civil wrong or injury to person or property. *Negligence* is a classification of tort in which the injury is not intentionally caused but results from some breach of a legal duty to exercise care (*1*). *Common law* refers to that body of law developed by the courts in the cumulative adjudication of individual cases and is to be distinguished from statutory law, which is enacted by legislative bodies. When the United States won its independence from England and created its own form of republican government, many of the judicial traditions and principles of English law were incorporated into the newly formed judiciary of the new nation. One of the notions carried into U.S. jurisprudence from its roots in English common law was that, as the source of all law, the government, or "sovereign," as an institution could not be sued, at least not without its consent.

Consequently up until the latter half of the twentieth century individuals who were injured through the negligence of state governments or government highway departments had no recourse under the law to seek redress for their injuries. The law in virtually every state provided that either the state and its agencies could not be sued or, if the state and its agencies were amenable to suit, they could not be held liable in tort. This notion of "sovereign immunity" provided an absolute defense to personal injury lawsuits brought against state governments and state highway departments.

Legal scholars have written a great deal about the development of the doctrine of sovereign immunity in the United States. It serves no purpose here to reiterate or review in great detail the English and U.S. case law dealing with the doctrine or to debate the question of whether the U.S. incarnation of the doctrine accurately mirrors sovereign immunity as it developed in the common law of Great Britain. A brief but excellent exegesis on the topic can be found in a work by Thomas (*2*).

For many decades the doctrine in the United States protected state governments and highway departments, as well as highway department employees, from the threat that errors in judgment, mistakes, or outright carelessness could result in a lawsuit or, worse, a money judgment tapping state highway revenues. In the decades following World War II, however, U.S. social mores, as reflected in the legislative governance and jurisprudence of the various states, began to demand greater accountability from government and government officials. The notion that a citizen injured through the carelessness or negligence of government action should have no recourse for that injury began to erode. An accountable government was responsible for the injuries that it caused. The image of the immune sovereign state began to fade.

The erosion process was not, of course, immediate. And because the law of negligence is a matter of common law as developed in each state, the process was varied in both pace and character. In some states the turnaround was dramatic.

Arizona was one of the first states to abolish the doctrine of sovereign immunity as applied to a highway department and highway officials. In the case of *Stone v. Arizona Highway Commission*, 93 Ariz. 384, 381 P2d 107 (1963), signs and pavement markings indicating that the roadway curved to the left had been left in place for several months following the completion of a new section of roadway that had eliminated the curve. A mother of two was killed and the father and children were injured when the vehicle driven by the father crossed the centerline of the road at the crest of a hill and struck a vehicle traveling in the opposite direction. The surviving family sued a number of highway offi-

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cials and employees as well as the highway contractor who had constructed the improvement. The lower courts had dismissed the case as to the state and the highway officials because of the doctrine of sovereign immunity, in accordance with well-settled Arizona case law. The family then appealed. In its opinion deciding the appeal, the Arizona Supreme Court abruptly overturned the existing line of cases and abolished the doctrine of sovereign immunity in Arizona.

In discarding the doctrine the Arizona Supreme Court noted that sovereign immunity had long been a solid fixture of Arizona jurisprudence, but stated: "We are of the opinion that when the reason for a rule no longer exists, the rule itself should be abandoned," *Stone v. Arizona Highway Commission*, 93 Ariz. 387, 381 P2d 109 (1963). In a lengthy discourse the Arizona Supreme Court set out the history of the evolution of the doctrine of sovereign immunity "from its medieval English background to [the] present day Arizona law," *Stone v. Arizona Highway Commission*, 93 Ariz. 388, 381 P2d 109 (1963). The court's sociological rationale for abandoning the rule, however, was made clear:

> In 75 A.L.R. 1196, a classic observation as to the sociological aspects of sovereign immunity appears which has since been quoted with approval in several jurisdictions; . . . "The whole doctrine of governmental immunity from liability for tort rests upon a rotted foundation. It is almost incredible that in this modern age of comparative sociological enlightenment, and in a republic, the medieval absolutism supposed to be implicit in the maxim, 'the king can do no wrong,' should exempt the various branches of the government from liability for their torts, and that the entire burden of damage resulting from the wrongful acts of the government should be imposed upon the single individual who suffers the injury, rather than distributed among the entire community constituting the government, where it could be borne without hardship upon any individual, and where it justly belongs." *Stone v. Arizona Highway Commission*, 93 Ariz. 338, 381 P2d n.1 (1963).

With greater or lesser intensity than the Arizona Supreme Court, the courts and legislatures of most of the other states have embraced the same or similar reasoning and have abolished or limited the immunity of state government against liability for injury caused by the negligence of state highway departments and their employees. Today only six states report that they retain sovereign immunity against suits in tort: Arkansas, Maine, Mississippi, North Dakota, Wisconsin, and Wyoming (*3*).

DESIGN IMMUNITY

Even among states that have abolished sovereign immunity and allowed citizens to sue their government for injuries sustained on the state's highways, some have retained immunity against lawsuits claiming defects related to the choice of the highway's design. Highway planning and design are viewed by those states as discretionary functions of government requiring a high-level balancing of many financial, engineering, and public policy considerations. The laws in those states, whether embodied in judicial case law or in an enactment of the state legislature, recognize that in order for public officials to appropriately exercise their judgment or discretion for the public good, they should be free from any threat that the exercise of that judgment or discretion will result in tort liability.

Although many states provide immunity for the discretionary activities of the planning and design of highways, once a design is chosen and constructed, the laws in those states may establish an operational legal duty to protect the traveling public if the resulting highway is flawed. For example if a highway is designed and constructed with a curve that cannot be negotiated at the posted speed limit, the immunity defense may defeat a lawsuit on the basis of allegations of faulty design of the curve, but immunity may not provide a defense against allegations of a failure to reduce the speed limit through the curve or otherwise adequately warn the traveling public of the hazard of the curve [see *St. Petersburg v. Collum*, 419 So2d 1082 (Fla. 1982)].

It is not the purpose of this writing to survey the law in the 50 states or explicate the nuances of sovereign immunity for highway design as they vary in each state's body of statutory and common law. A comprehensive annotation accomplishing that survey and explication has been published elsewhere (*4*).

SIGNIFICANCE OF DESIGN STANDARDS

Notwithstanding the various statutory enactments in a number of states dealing with sovereign immunity, the law of negligence or tort has largely developed in the common law, that is, the body of law developed by judicial opinion, as the trial-level and appellate courts of each state adjudicate individual cases brought before them. And because each state's judicial system is independent and unique to that state, the law varies in some degree from the courts of one state to another. Nevertheless the basic elements of a tort or negligence lawsuit are common in jurisprudence throughout the United States. A fundamental understanding of those elements is necessary to discuss the legal importance of design standards and the legal implications of exceptions to those design standards.

The basic elements of a negligence lawsuit are

1. The existence of a legal duty of the defendant toward the plaintiff,
2. A breach of or failure in that duty by the defendant,
3. Injury to the plaintiff's person or property, and
4. A causal connection between the breach or failure of duty and the injury.

In the most general, oversimplified terms, the legal duty of any defendant toward a plaintiff is to exercise a level of care in acting that is reasonable under the circumstances of the particular given case. Consequently the standard by which a breach of duty is measured is commonly referred to as the *reasonable care* standard, that is, whether an ordinary person faced with the circumstances evident in a particular case might reasonably have acted with the same degree of care as the defendant. If the jury decides that a reasonable person would have exercised more care than the defendant, a breach of the duty has been found.

As noted at the outset of this writing, the law in most states recognizes some legal duty in highway departments and highway officials to provide safe and convenient highways for the traveling public. In those states not providing for design immunity, the state highway department's legal duty includes a duty to plan and design highways in a manner that provides for the safety of the public that uses them. In simplified terms highway departments and officials in all but a handful of states still retaining sovereign immunity must exercise reasonable care in the construction and maintenance of the state's highways. In the states without design immunity they must exercise a reasonable level of care in the planning and design of highways as well.

In judging the reasonableness of ordinary behavior, the jury in a typical tort case relies on its individual and collective everyday experience to determine the appropriate standard of care to which a defendant is to be held. In more complex cases, however, those involving medical malpractice, for example, the jury has no direct knowledge of what the standard of care might be in a particular circumstance. It must be educated as to what the standard of practice in a particular medical field was at the time of the injury. It must rely on information from outside the experience of the individual jury members.

Such is the case as well in lawsuits involving alleged defective highway design. The jury, unless it is composed entirely of highway engineers, would not know whether a particular stretch of highway was properly designed without reliance on some information outside its own experience. The jury would have to be educated as to the applicable design standards. As in cases involving other professional fields, that information would be provided by expert witnesses, who in turn might rely upon treatises, practice guidelines, or other texts to support their opinion of what the applicable standard of care was.

In highway design cases, the texts relied upon would likely be AASHTO guidelines, federal and state regulations and guidelines, and perhaps papers and publications issued by organizations such as TRB.

Design standards therefore provide the benchmark for the jury to determine whether the agency or individuals designing the highway exercised reasonable care in the development of the highway plan and the configuration of the highway geometry. In general if the applicable standards are followed, reasonable care was exercised and no breach of duty occurred. Conversely if applicable standards were not followed, perhaps reasonable care was not exercised and a breach of the highway agency's duty has occurred.

For all the precision in the field of highway engineering, however, the practical applicability of a particular highway design standard in a given set of circumstances is not always easily determined. In the law, regardless of the standard for practice established by the expert witnesses and their texts, it is the appropriateness of adherence to the standard in the narrow, particular circumstance of the facts of the individual case being tried that the jury must determine. Deviation from a standard does not per se establish that reasonable care was not exercised. And adherence to the standard does not per se establish that reasonable care was exercised. As it is for the engineering practitioner, each design standard is, for the jury in a highway design tort case, but a single criterion that must be considered in the total context of the numerous and variable factors brought to bear in each individual case.

DESIGN EXCEPTIONS

With design standards as the benchmark, then, the implications of a deviation from the standards must be examined.

As noted above the deviation from a design standard does not constitute negligence as a matter of law, and adherence to standards does not, in every case, mean that the appropriate standard of case has been met. Many of the published standards themselves are couched in terms of providing a guideline rather than a hard-and-fast rule, so that whether the deviation from the standard constituted a failure to exercise reasonable care becomes one for the jury to struggle with and ultimately decide. Design standards are also frequently phrased as a minimum value. The standard may indicate, for example, that the highway geometry under particular conditions should provide "at least" a specified length for the curve radius. The standard is simply a minimum measure below which a particular design should not fall. The experienced plaintiff's counsel may present expert engineering testimony that the minimum standard was not sufficient to adequately provide for the safety of the traveling public. Consequently, even though the design standard was met, the jury might still find that the design did not constitute the exercise of reasonable care.

Adherence to the design standards applicable in the field of highway engineering at a particular time will, however, often give rise to a presumption that no negligence occurred in the design of the highway. The presumption may arise as a matter of law (statutory law or case law may expressly provide that conformance to a standard gives rise to a presumption of no negligence, and the jury will be so instructed by the trial judge prior to its deliberation) or may simply form in the minds of the jurors from their common understanding (or misunderstanding) of the significance of standards. In any case the presumption may be successfully rebutted by evidence that the highway was nevertheless known to be unsafe for the volume or class of vehicles using it [see *Hampton v. State Highway Commission*, 209 Kans. 565, 498 P2d 236 (1972)].

Similarly the deviation from a design standard may, in some state courts, give rise to a presumption of negligence on the part of the highway department or designer as a matter of law (see above). This may be especially true if the standard violated was found in a state regulation. The deviation from an accepted engineering standard will constitute at the very least evidence that the appropriate standard of care in the design of the highway may not have been met and will probably give rise to a presumption in the minds of the jurors that negligence has occurred. To rebut the presumption or counter the evidence, it is necessary for the defendant highway department or designer to present persuasive evidence either that the standard put forth by the plaintiff was not applicable to the circumstances at hand or that the standard could not reasonably be met.

It is one of the fundamental purposes of the design exception process to provide justification for deviation from accepted design standards that might otherwise be applicable in the design of a particular highway project. The focus of those procedures, however, is not specifically the defense in court of tort lawsuits that might potentially arise because of accidents and injury alleged to have resulted from the design of the highway project. FHWA, however, in adopting highway design standards and providing for exceptions to those standards, has articulated as its goal:

> [T]o provide the highest practical and feasible level of safety for people and property associated with the Nation's highway transportation systems and to reduce highway hazards and the resulting number and severity of accidents on all the Nation's highways [23 C.F.R. 625.2(c)].

Consistent with this articulated goal, the provisions allowing for departure from the standards state:

> The determination to approve a project design that does not conform to the minimum criteria is to be made only after due consideration is given to all project conditions such as maximum service and safety benefits for the dollar invested. [23 C.F.R. 625.3(f)(2)].

Consequently, a departure from the minimum design standards that is shown to be justified in accordance with the procedures for exception established by FHWA may persuade the jury that reasonable care was taken in the choice of highway design.

RECOMMENDATIONS

In the quest to avoid or minimize the exposure to tort liability that may arise in the design exception process, the best course obviously is to meet or exceed the applicable standards in the design of the project. Citizens who may bitterly criticize the performance of their state highway department and its employees show surprisingly great respect for the engineering standards developed in the transportation industry when they sit as a jury. And the presence of the deep pocket of a government does not outweigh the recognition that the transportation engineering industry is a highly sophisticated and professional one whose standards have been developed from considerable study and experience.

It is equally obvious that if a deviation from design standards is necessary, the deviation should be minimized. In its task of determining whether reasonable care was exercised, a jury is more likely to be forgiving of a small rather than a great deviation and of a single rather than a number of design exceptions.

When a deviation from design standards is necessary and application for design exceptions must be made, it is important, from a tort defense perspective, that the justification given be complete. Although a lengthy explanation of the factors requiring the deviation from the design standards may not be required to satisfy FHWA, the more engineering justification available for the jury to consider, the greater the persuasive weight that the departure from the standard was nevertheless an exercise of reasonable care. If the justification provided succinctly but completely describes the physical and perhaps environmental factors that make the exception necessary, a jury is likely to be persuaded that the judgment to depart from the standard was nevertheless sound. Justification in economic rather than engineering terms is not recommended. In some state courts, evidence of economic justification is not permitted to try to convince a jury that reasonable care was exercised in the project design. Such is the case in Pennsylvania. In any state court a jury faced with an injured plaintiff is less likely to be persuaded by an argument that a design exception was needed to save tax dollars than by justification rooted in sound engineering judgment.

In addition to a complete description of all the factors justifying the exception, it may also be helpful, from a tort defense perspective, to include language that leaves open the door to further justification by an engineering expert, if the design exception becomes the subject of a tort lawsuit. As noted earlier the parties in a design-related lawsuit will try to persuade the jury as to whether the appropriate standard of care was met with the use of highway engineering experts. It can be helpful if the expert testifying on behalf of the highway department or highway designer has a file that not only completely describes the reasoning of the engineer who initially justified the exception but that also indicates that there may be some other engineering justification for the exception that was not detailed. Simple language, such as "among the factors justifying the exception are the following" or "some of the considerations requiring the approval of the design exception are," affords the expert engineering witness the opportunity to expand on the justification initially offered, if the matter is raised in the trial of a tort suit.

Other documentation reflecting not only that the exception was justified after consideration of engineering difficulties and other factors but noting that the resulting project design was nevertheless a safe one can also be helpful in the defense of a tort suit. In this regard review and approval of the exception, with such notation, by several engineers may demonstrate that in the sound engineering opinion of more than one professional the design is a reasonable one. Approval by FHWA officials itself may also offer some persuasive weight.

CONCLUSION

To the extent that the tort liability experience of Pennsylvania is typical of that found in states whose highway departments and highway designers do not enjoy design immunity, it must be noted that the number of lawsuits in which a design defect is alleged to be the cause of an accident and the resulting injury is not great in comparison with the number of cases alleging faulty signing or faulty maintenance. And the number of lawsuits in which the alleged negligence involves a design exception is smaller still. Highway engineering and design is a sophisticated and highly professional field. Its practitioners hold themselves to demanding standards that are continually reevaluated and raised. The high standard of professionalism in the field is reflected in the relatively few lawsuits relating to design and the design exception process.

Consequently it is not the intent of this writing to suggest that the primary focus of the highway designer, in seeking the approval of design exceptions, should be the avoidance of tort liability or the preparation of a file for use at trial. But with awareness of potential liability, in those states where it is a reality, the designer can, remembering the issues discussed here, vindicate the high standards of the profession if the sound design decisions are challenged in court.

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Accident Relationships of Roadway Width on Low-Volume Roads

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An analysis was performed to quantify the accident effects of lane and shoulder widths on rural roads carrying fewer than 2,000 vehicles per day. The primary data base used in the research contained accident and roadway characteristic information for more than 6600 km (4,100 mi) of two-lane roadway sections in seven states. Independent data bases from three states (Minnesota, Illinois, and North Carolina) for roadways totaling more than 86 000 km (54,000 mi) were selected to validate the accident relationships found in the primary data base. Analysis of covariance was used to quantify accident relationships on these low-volume roads. Single-vehicle and opposite-direction accidents were classified as *related* accidents because the accident rates for these two types were found to be related to differences in lane and shoulder widths. The rate of related accidents was also affected by roadside hazard, roadway terrain, the number of driveways per mile, and state differences. No differences in accident rates were found between roadways with paved and unpaved shoulders. For lane widths of at least 3.0 m (10 ft), related accident rates were lower when wide shoulders were present than when narrow shoulders were present. For a given shoulder width, wider lanes were found to be associated with lower accident rates. Somewhat counterintuitively the accident rate was higher for 3.0-m (10-ft) lanes with narrow shoulders than for 2.7-m (9-ft) lanes with narrow or wide shoulders. For traffic volumes of 250 vehicles per day or less, accident rates did not differ significantly between paved and unpaved roads. For traffic volumes of greater than 250 vehicles per day paved roads have significantly lower accident rates than unpaved (dirt and gravel) roads. The research findings indicate that on low-volume roads lane widths as narrow as 2.7 m (9 ft) may be acceptable from a safety standpoint under certain conditions. The 1995 draft AASHTO policy chapter on local roads includes revised roadway width guidelines that reflect many of the research findings presented.

Increasing concern has been expressed by safety professionals in recent years regarding the safety of low-volume roads [e.g., roads carrying fewer than approximately 2,000 vehicles per day (vpd)], since such roads constitute a major portion of the U.S. highway network. For example, of the 5.0 million km (3.1 million mi) of all two-lane rural roads, approximately 90 percent have average daily traffic (ADT) of less than 1,000 vpd. About 80 percent have ADT of less than 400 vpd, and 38 percent carry fewer than 50 vpd. Considering only the local and minor collector roads on the two-lane system, 90 percent have ADT of 2,000 vpd or less; more than 60 percent of minor rural arterials have ADT of 2,000 vpd or less (*1*).

Maintenance and reconstruction of the two-lane highway system have emerged as serious problems not only because of the extensive size of the system but also because significant portions of two-lane highways were designed and built to outdated standards not reflective of current design policy. For example, over one-quarter of the mileage of such roads have lane widths of 2.7 m (9 ft) or less, and two-thirds have shoulder widths of 1.2 m (4 ft) or less. In addition, 11.5 percent of two-lane highway mileage has no shoulders (*1*). These statistics are in contrast to the current design values given in the 1990 AASHTO policy, *A Policy on Geometric Design of Highways and Streets* (*2*). For all but extremely low-volume and low-speed highways, the current policy calls for 6.7- to 7.3-m (22- to 24-ft) roadways regardless of terrain or other conditions (*2*). Also a large portion of low-volume roads is unpaved, which presents maintenance problems in addition to safety concerns.

Controversy has existed over the optimal lane and shoulder widths for these low-volume roads with respect to whether existing roadways should be widened or new roadways constructed. Such decisions require the availability of quantifiable accident relationships on roadways with various lane and shoulder widths and types. Although numerous safety studies have been conducted in the past decade to address the safety effects of lanes and shoulders, few have focused exclusively on low-volume roads. Such an analysis was the focus of this study.

BACKGROUND SAFETY RESEARCH

During the past 25 years dozens of studies concerning the relative safety of various roadway widths have been conducted. One of the most comprehensive and more recent studies conducted to date on the safety effects of roadway width was a 1987 study by Zegeer et al. (*3*) for FHWA that involved an analysis of 7971 km (4,951 mi) of two-lane roadways in seven states. It included 7704 km (4,785 mi) of rural road and only 267 km (166 mi) of urban roadway. Accident prediction models were used to determine the expected accident reductions related to various geometric improvements. Accident types found to be most related to cross-sectional features (e.g., lanes, shoulders, and roadside condition) included run-off-the-road, head-on, and sideswipe (same direction and opposite direction) accidents. The roadway variables found to be associated with a reduced incidence of these related accident types were wider lanes, wider shoulders, better roadside condition, flatter terrain, and lower traffic volume (*3*).

For lane widths of from 2.4 to 3.7 m (8 to 12 ft), the predictive accident model showed that related accidents were reduced by approximately 12 percent for each 0.3 m (1 ft) of lane widening. For shoulder widths of between 0 and 3.7 m (0 and 12 ft), the percent reduction in related accidents as a result of the widening of paved shoulders ranged from 16 percent [for 0.6 m (2 ft) of widening] to 40 percent [for 1.8 m (6 ft) of widening]. Paved

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shoulders were slightly safer than unpaved shoulders. However approximately half of the roadways in that study sample had ADT of more than 2,000 vpd, there were no unpaved roads, and a minimal sample of roads with ADT values of less than 750 vpd was available (*3*).

Note that the results of that study (*3*) showing a constant percentage reduction for each foot of lane or shoulder widening are somewhat counterintuitive. That is, one might expect that widening of lanes from 2.4 to 2.7 m (8 to 9 ft) would result in a higher percentage reduction in accidents than widening from 3.4 to 3.7 m (11 to 12 ft). Although the model forms found in that study did not show this, it should be mentioned that the net number of accidents reduced would be greater for widening narrow [e.g., 2.4-m (8-ft)] lanes than for widening wider [e.g., 3.4-m (11-ft)] lanes, since, for example, the accident rate in the before condition is greater for 2.4-m (8-ft) lanes than for 3.4-m (11-ft) lanes. Thus a 12 percent accident reduction [per 0.305 m (1 ft) of widening] would represent more net accidents reduced on a road with narrow lanes (and a higher accident rate) than on a road with wider lanes.

A study that addressed low-volume rural roads in one state was a 1988 study by Griffin and Mak (*4*) that attempted to quantify the relationship between accident rate and roadway surface width on two-lane rural roads in Texas with ADTs of 1,500 vpd or fewer. Log-linear accident prediction models were developed for 58 306 km (36,215 mi) of roadway within several ADT categories. Multivehicle accident rates [number of accidents per 1.61 km (1 mi) per year] were not found to be related to surface width for any of the ADT groups tested. Single-vehicle accident rates were found to increase as roadway width decreased for ADT groups of between 401 and 1,500 vpd. Accident reduction factors were developed for various widening projects within these ADT ranges, and those accident reductions matched closely with those in the study of Zegeer et al. (*3*). On the basis of an economic analysis, widening was not found to be cost-beneficial for ADT values of less than 1,000 vpd (*4*).

Numerous other studies have also analyzed large state data bases to determine accident effects of lane and shoulder widths. These include studies by Foody and Long (*5*) in Ohio, Zegeer et al. (*6*) in Kentucky, Shannon and Stanley (*7*) in Idaho, and an NCHRP study by Jorgensen, Roy & Associates with data from Washington and Maryland (*8*), among others. Although those studies used a wide range of sample sizes and analysis techniques, all basically found that accident rates decreased as a result of wider lanes or shoulders, even though there was considerable variation in the exact amount of crash reduction.

Studies by Rinde (*9*) (California) and Rogness et al. (*10*) (Texas) involved evaluations of actual pavement-widening projects. Those results supported the findings in the other studies in terms of the beneficial effects of lane and shoulder widening, the types of crashes reduced, and the relative magnitudes of the effects of widening. A 1974 study by Heimbach et al. (*11*) in North Carolina also found that paving 0.9- to 1.2-m (3- to 4-ft) unpaved shoulders results in significant reductions in accident frequency and severity.

RESEARCH OBJECTIVE AND APPROACH

Although past research laid the groundwork for what is currently known on the subject, there was a need to look more closely at accident relationships for low-volume roads only, including paved and unpaved roads, and for roads in a variety of functional classifications (arterial, collector, and local) with varying roadway conditions, and to do so with a sample that included data from more than a single state. Also there was a need to determine what specific traveled way and shoulder width combinations provide reasonable levels of safety for various conditions.

The objective of the study was to quantify the accident effects of lane width, shoulder width, and shoulder type for a variety of traffic and roadway conditions for rural roads with traffic volumes of 2,000 vpd or fewer. Although ADT of 2,000 vpd or less does not constitute an official definition of low volume, it is the value used in AASHTO design guidelines for roadway width (*2*) and was chosen for use in the analysis in the present study. The study also involved an investigation of the safety of paved versus unpaved roadway surfaces for these lower-volume roads.

A detailed statistical analysis was conducted on a primary data base of approximately 6600 km (4,100 mi) of low-volume, two-lane roads in seven states. Adjusted accident rates were determined for various lane and shoulder widths by analysis of covariance. To validate and investigate these relationships further, three additional independent data bases for roadways totaling more than 87 000 km (54,000 mi) of low-volume, two-lane roads from three states (Illinois, Minnesota, and North Carolina) were analyzed. These validation data bases from Illinois and Minnesota were part of FHWA's Highway Safety Information System (HSIS), which consists of computerized accident, traffic, and roadway data files from five states. The accident effects of other roadway variables were also determined from the analysis. Note that the validation data bases did not include information on level of hazard of the roadside and did not include any sections used in the primary data base.

SELECTED DATA COLLECTION VARIABLES

Roadway and Traffic Variables

Crash experience on rural highways is a complex function of many factors, including those associated with physical aspects of the roadway, and many other factors related to driver, vehicle, traffic, and environmental conditions. On the basis of their relationships to accidents developed in past research, the traffic and roadway variables selected for data collection included

- Section information (section identification and length);
- Pavement type (paved or unpaved);
- Lane width, shoulder width, and type of shoulder (i.e., paved, gravel, or earth);
- General terrain (i.e., flat, rolling, or mountainous);
- Type of area and development;
- Design speed;
- Functional roadway class;
- Number of driveways (per kilometer or mile);
- Number of intersections (per kilometer or mile);
- Percent trucks;
- Speed limit;
- Average annual daily traffic (AADT);
- Horizontal alignment (i.e., percentage of the section with a curvature of greater than 2.5 degrees);
- Vertical alignment (i.e., percentage of the section with a grade of greater than 2.5 percent);

- Side slope ratio (2:1 and steeper, 3:1, 4:1, 5:1, 6:1, or 7:1 and flatter); and
- Measures of general roadside hazard (see below).

The two measures of roadside hazard used in the data collection and analysis were termed *roadside recovery distance* and *roadside hazard rating*. These measures were used in the 1987 FHWA study by Zegeer et al. (*3*) on two-lane rural roads and were both found to have a significant relationship to accidents. The ratings for the roadside hazard rating used in that study (and the current study) are based on a seven-point pictorial scale for rural highways. The data collectors chose the rating value (one through seven) that most closely matched the general roadside hazard level observed beside the roadway section in question.

In addition to the subjective roadside hazard rating, a measure termed *roadside recovery distance* also was determined for each section. This measure is relatively similar to the definition of a *clear zone*, in that it is the lateral distance from the edgeline (i.e., outer edge of the traffic lane) to the closest object that would cause a fixed-object or rollover collision, that is, the closest lateral distance to trees, utility poles, culvert head wall, bridge rail, steep slope (i.e., steeper than 3:1), and so on. Thus like the roadside rating, the roadside recovery distance basically measures the degree of forgiveness of the roadside.

Accident Variables

Although dozens of accident variables could have been chosen for analysis purposes, only those necessary for the analysis were selected. For each roadway section accident information included:

- Years of crash data (5 years in each case);
- Total number of accidents on the section;
- Number of accidents by severity (property damage only, A injury, B injury, C injury, and fatality);
- Number of people killed;
- Number of crashes by light condition (daylight or darkness);
- Number of accidents by pavement conditions (dry, wet, or icy); and
- Number of crashes by type (fixed-object, rollover, other run-off-the-road, head-on, opposite-direction sideswipe, same-direction sideswipe, rear-end, backing or parking, pedestrian or bike or moped, angle or turning, train-related, animal-related, and other or unknown types).

Selection of the Data Base

The data sample selected for analysis was a computer file consisting of sections of two-lane roads, each with its corresponding roadway, traffic, and accident characteristics. This type of data base allows a comparison of the accident experience associated with different roadway widths, paved versus unpaved roadway surfaces, and other roadway features. Ideally each roadway section should be of sufficient length to allow for calculation of accident rates in terms of the number of accidents per 1.61 million vehicle km [accidents per million vehicle mi (MVM)]. Section lengths of 1.61 km (1 mi) or greater were generally chosen to help ensure adequate crash data and thus stability of the rates, since very short sections can yield unstable accident rates. Note that even with these longer section lengths some of the low-volume sections had no accidents in the 5-year analysis period.

Sample size requirements were computed to enable detection of at least a 10 percent difference in accident rate between roadway width groupings at a significance level of 0.05 (i.e., a 95 percent confidence level). The analysis revealed that a sample of at least 4025 km (2,500 mi) would be adequate. Ultimately a sample of 6661 km (4,137 mi) was available for use in the primary analysis. Independent samples of roadway sections also were used to validate these accident relationships, as discussed later.

The bulk of the data came from the data base on two-lane rural roads developed for TRB and FHWA in the study *Safety Effects of Cross-Section Design for Two-Lane Roads* (*3*). The data base developed for that earlier effort is perhaps the most complete multistate data base on two-lane roads in terms of roadway section representation, the amount of data sampled, and the wide variety of accident, traffic, roadway, and roadside variables for which data were collected.

The data base consists of a sample of 7971 km (4,951 mi) on paved, two-lane roads from Alabama, Michigan, Montana, North Carolina, Utah, Washington, and West Virginia. Perhaps the most pertinent data variables collected in that study that are not available from standard state accident or roadway inventory files were those related to side slope and roadside hazard. However the FHWA cross-section data base provided only approximately 4300 km (2,700 mi) with ADT values of 2,000 vpd or less. Also it had no samples of unpaved roads and inadequate samples of roads with a local functional class and within a very low ADT range (particularly ADT values of less than 750 vpd). Thus other data sources were needed to fill these gaps.

Three state or local data bases (North Carolina, Utah, and Oakland County, Michigan) were selected to supplement the cross-section data base. Selection of additional sections in three of the seven cross-section states reduced the level of introduction of additional state biases resulting from different state reporting thresholds, state coding practices, or other factors. The wide variety of climates, driver characteristics, roadway design practices, and other factors contained within the seven states helped to ensure a diverse sample of roadway and traffic conditions. Within the three state or local data bases, roadway sections were selected as needed to fill the data gaps. The final primary data base thus contained 1,277 roadway sections with a total of 6661 km (4,137 mi) including 895 km (556 mi) of unpaved roads and 5765 km (3,581 mi) of paved roads. The average section length was 5.2 km (3.2 mi).

STUDY FINDINGS

Issue 1: Characteristics of Accidents on Low-Volume Roads

The question of most interest was how accidents on rural, low-volume roads differ from accidents on similar roads with higher volumes. The accident characteristics were first determined for the 5-year sample of 14,888 accidents that occurred on the 6661 km (4,137 mi) of low-volume roads, termed the *primary data base*, analyzed in the study. This was then compared with the full rural sample of 62,676 crashes on the 7704 km (4,785 mi) of rural two-lane roads in the data base from the earlier FHWA study (with a full range of ADT, including low-volume roads). With respect to overall rates, the average accident rate for the total data base for low-volume roads was 3.5 accidents per 1.61 million vehicle km (MVM) in comparison with an overall rate of 2.4 accidents per 1.61 million vehicle km (MVM) for the higher-volume full sample.

With respect to accident types, a greater percentage of fixed-object crashes, rollover crashes, and other run-off-the-road crashes occurred on low-volume roads than on the full sample of rural roads (Table 1). Conversely the data showed a lower percentage of crashes involving rear-end collisions and angle and turning collisions for low-volume roads. This may be expected, because there are fewer other vehicles to strike on low-volume roads than on higher-volume routes.

Issue 2: Determining Related Accident Types

Analysis of covariance models were used to identify accident types that are associated with roadway width. The independent roadway variables included lane width, shoulder width, terrain, and roadside hazard rating. Accident rates were found to be significantly associated with varying lane and shoulder widths for single-vehicle accidents and opposite-direction accidents. Rates of other accident types (angle, turning, etc.) were found not to be significantly related to lane or shoulder width. These findings agree closely with the 1987 study by Zegeer et al. (*3*) of rural, two-lane roads with all ADT ranges. However, that study not only related single-vehicle and opposite-direction accidents to roadway width but also found that same-direction sideswipe accidents were marginally significant, the latter finding was not confirmed in the present study for low-volume roads. In all of the remaining analyses, single-vehicle and opposite-direction accidents were combined and are referred to as *related accidents.*

Issue 3: Important Traffic and Roadway Variables

The traffic and roadway variables found to be significantly related to the rate of related accidents included

- Lane and shoulder width (or total roadway width);
- Roadside hazard rating and roadside recovery distance;
- Number of driveways per 1.61 km (1 mi);
- Terrain; and

TABLE 1 Summary of Accident Types and Characteristics for Low-Volume Road Sites

Accident Type	Primary Database on Low-Volume Roads		Cross-Section Database	
	Number of Accidents	Percent of Total Accidents	Number of Accidents	Percent of Total Accidents
Total	14,888	100.0	62,676	100.0
Property Damage Only	8,973	60.3	38,857	62.0
Injury	5,632	37.8	22,944	36.6
Fatal	283	1.9	875	1.4
Injuries*	8,768	N/A	37,321	N/A
Fatalities*	328	N/A	1,068	N/A
Daylight	8,050	54.1	37,402	59.7
Dawn/Dusk	820	5.5	2,888	4.6
Dark with Lights	160	1.1	2,770	4.4
Dark without Lights	5,809	39.0	19,496	31.1
Light Unknown	49	0.3	120	0.2
Dry	10,306	69.2	41,957	66.9
Wet	2,442	16.4	13,487	21.5
Snow/Ice	1,952	13.1	6,657	10.6
Unknown Pavement	188	1.3	575	0.9
Run-Off-Road - Fixed Object	4,017	27.0	12,091	19.3
Run-Off-Road - Rollover	1,999	13.4	4,245	6.8
Run-Off-Road - Other	2,287	15.4	2,840	4.5
Head-On	475	3.2	2,113	3.4
Opposite Direction Sideswipe	642	4.3	2,997	4.8
Same Direction Sideswipe	330	2.2	2,288	3.7
Rear-End	893	6.0	12,420	19.8
Parking/Backing	264	1.8	1,155	1.8
Ped/Bike Moped	117	0.8	655	1.0
Angle & Turning	1,773	11.9	14,730	23.5
Train	20	0.1	47	0.1
Animal	1,404	9.4	5,212	8.3
Other or Unknown	667	4.5	1,883	3.0

*The data for these variables represent the number of people injured or killed, and not the number of accidents.
N/A = Not applicable.

- State (grouped with respect to similar related accident rates): (a) Alabama, Montana, and Washington, (b) North Carolina and Michigan, and (c) Utah and West Virginia.

Variables for percent grade and curvature were not considered for further analysis, since they were available for only about half of the study sections. Instead the terrain variable was significant and served as a general measure of alignment for use as a control variable. The functional class variable was found to relate highly to roadway width (i.e., higher functional classes generally have wider roads) and state (i.e., some states tended to assign the same one or two functional class categories to all their low-volume roads, but such designations differed from state to state).

Variables that were found to not be associated significantly with accidents on low-volume roads were the number of intersections per 1.61 km (1 mi) (i.e., most sections had no major intersections), speed limit [i.e., most sections had 89-km/hr (55-mph) speed limits whether posted or not, regardless of the alignment or design speed], and the percentage of trucks (i.e., very few of the sections had a substantial volume of heavy trucks). The formulation of accident models was sensitive to these relationships.

It is also interesting to note that shoulder type (i.e., paved versus unpaved shoulders) was not found to affect significantly the number of accidents on low-volume roads. The 1987 study by Zegeer et al. (*3*) did find a small but significant reduction in the number of accidents on roadways with paved shoulders in comparison with the number on roadways with unpaved shoulders for a full range of traffic volumes. These findings may indicate that shoulder paving is more beneficial on higher-volume routes (e.g., those with more larger trucks) than on lower-volume routes.

Issue 4: Accident Effects of Lane and Shoulder Width on Paved Roads

Covariance models were used to estimate rates of related accidents as a function of lane and shoulder width while adjusting for roadside hazard rating, terrain, state, and the number of driveways per 1.61 km (1 mi). The following discussion of lane and shoulder width effects pertains only to paved roads on which shoulders are either paved or unpaved. The lane and shoulder width refer to the average width on one side. For example a shoulder width of 1.8 m (6 ft) refers to a 1.8-m (6-ft) shoulder on each side of the road. Because shoulder type was not found to significantly affect accident rate on low-volume roads, the shoulder width used in these analyses corresponds to the total width of each shoulder, regardless of the shoulder type. Unpaved roads will be considered later.

The results revealed that lane width and shoulder width each has a significant effect on the related accident rate. Six lane width categories [≤2.4, 2.7, >4.0 m (≤8, 9, 10, 11, 12, ≥13 ft)] and five shoulder width categories 0, 1 to 2, 3 to 4, 5 to 6, and >6 ft (0, 0.3 to 0.6, . . . >1.8 m) were used. Some analyses were conducted for various combinations of lane and shoulder widths, termed *total roadway width.*

Two separate models were developed for related accident rate by total roadway width (Figure 1). Model I represents the estimated rate of related accidents for various widths of roadway (i.e., lanes plus shoulders) while controlling for state, terrain, roadside recovery distance, and number of driveways per 1.61 km (1 mi). For Model II state, functional class (local versus all others), terrain, roadside hazard rating, and the number of driveways per 1.61 km (1 mi) were included as independent variables.

Both models have the same general shape, in which the rate of related accidents tends to decrease as roadway widths increase from 6.1 to 9.8 m (20 to 32 ft). However the rate for the most narrow roadway widths [5.5 m (18 ft) or less] was much lower than that for most wide roadways. Also no clear accident reduction was found for roadway widths of greater than 9.8 m (32 ft).

Because the models for total width do not provide details on the interaction of lane width with shoulder width, rates of related accidents were determined for various categories for lane and shoulder widths, as shown in Figure 2. Lane and shoulder width groupings were determined on the basis of the available sample sizes and by consideration of when significant accident differences exist. Data for only 134 km (83 mi) of roads with 2.4-m (8-ft) lanes were available, so a reliable accident rate could not be determined for roadways with that lane width. The resulting rate of related accidents for 2.7-m (9-ft) lanes was 1.69 accidents per 1.61 MV km (MVM) for shoulders of 1.2 m (4 ft) or less, and a rate of 1.56 for shoulders of 1.5 m (5 ft) or greater. Thus on roads with 2.7-m (9-ft) lanes, accident rates were not affected by wider shoulders.

One possible explanation for these findings is that vehicle speeds are lower on roads striped with 2.7-m (9-ft) lanes than on roads with wider lanes, regardless of the shoulder width. Somewhat unexpectedly the accident rate of 1.69 for roads with 2.7-m (9-ft) lanes with narrow [(0- to 1.2-m 0- to 4-ft)] shoulders was lower than the rate of 2.41 for roads with 3.1-m (10-ft) lanes with narrow shoulders. Roads with wider shoulders [greater than 1.5 m (5 ft)] and with 3.1-m (10-ft) lanes had lower accident rates (1.43), as shown in Figure 2. Further review of accident rates from several validation data bases was helpful in further examination of this somewhat surprising finding, as discussed later. No significant difference in accident rate was found between roads with 3.4- and 3.7-m (11- and 12-ft) lane widths, so data for roads with these lane widths were grouped together. The accident rate for roads with 4.0-m (13-ft) lanes and narrow shoulders was slightly lower (1.57) than the rate of 1.87 for roads with 3.3- and 3.7-m (11- and 12-ft) lanes.

Note that shoulder width categories were determined on the basis of actual accident rate differences and not set arbitrarily. Thus in terms of lane width effects, the initial analysis revealed that low-volume roads with 3.1-m (10-ft) lane widths with narrow or no shoulders have higher accident rates than low-volume roads with 2.7-m (9-ft) lane widths (of any shoulder width). Furthermore, for sections with narrow shoulders, accident rates were significantly lower for 3.4- and 3.7-m (11- and 12-ft) lanes than for 3.1-m (10-ft) lanes. Although roads with 4.0-m (13-ft) lanes with narrow shoulders had slightly lower accident rates than those with 3.3-and 3.7-m (11- and 12-ft) lanes, the sample size of roads with 4.0-m (13-ft) lanes with wide shoulders was small. Also, the practicality of providing 4.0-m (13-ft) lane widths for low-volume roads is questionable, and thus, 4.0-m (13-ft) lane widths were not considered further in the present study.

Validation of Analysis Results

The lower accident rate for roadways with 2.7-m (9-ft) lanes was unexpected and open to question and thus warranted further investigation with additional data bases of paved, low-volume roads

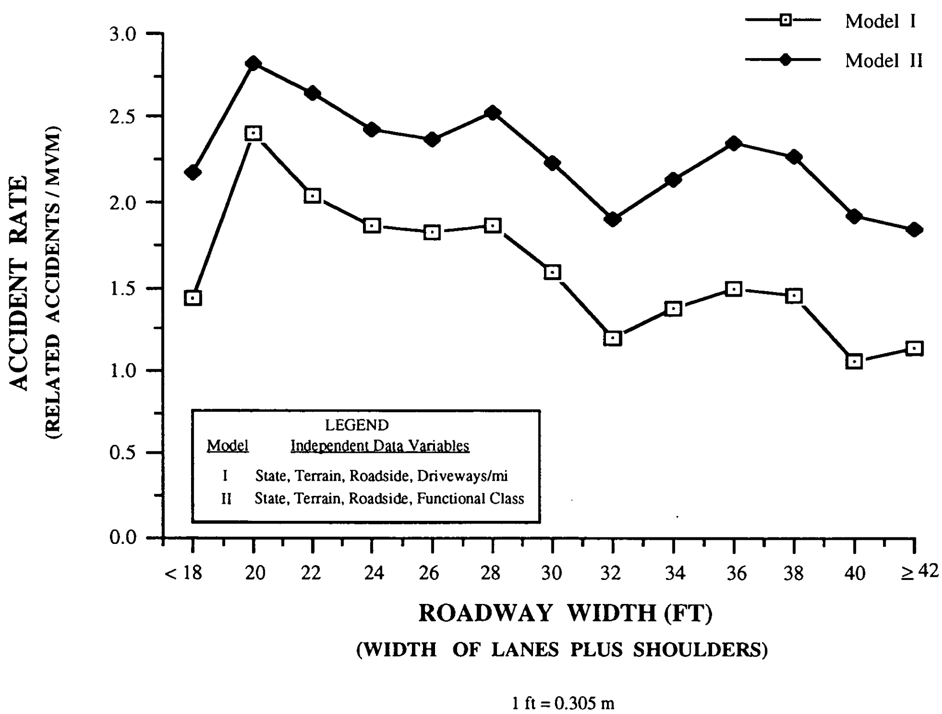

FIGURE 1 Rates of related accidents by roadway width from Models I and II.

from three states: Illinois [6104 km (3,791 mi)], Minnesota [39 121 km (24,299 mi)], and North Carolina [22 022 km (13,678 mi)]. Although detailed data on clear zone-roadside hazard were not available in these data bases, the other important variables were available.

On the basis of analysis of covariance models accident rates were computed for various lane and shoulder widths for the Illinois and Minnesota data bases, as shown in Figure 3. As was found with the primary data base, accident rates were again found to be quite low for roads with 2.7-m (9-ft) lanes and increased for roads with 3.0-m (10-ft) lanes with narrow shoulders. Accident rates were considerably lower on roads with 3.0-m (10-ft) lanes with wider shoulders and leveled off for roads with lane widths of 3.3 and 3.7 m (11 and 12 ft). These results confirm the results

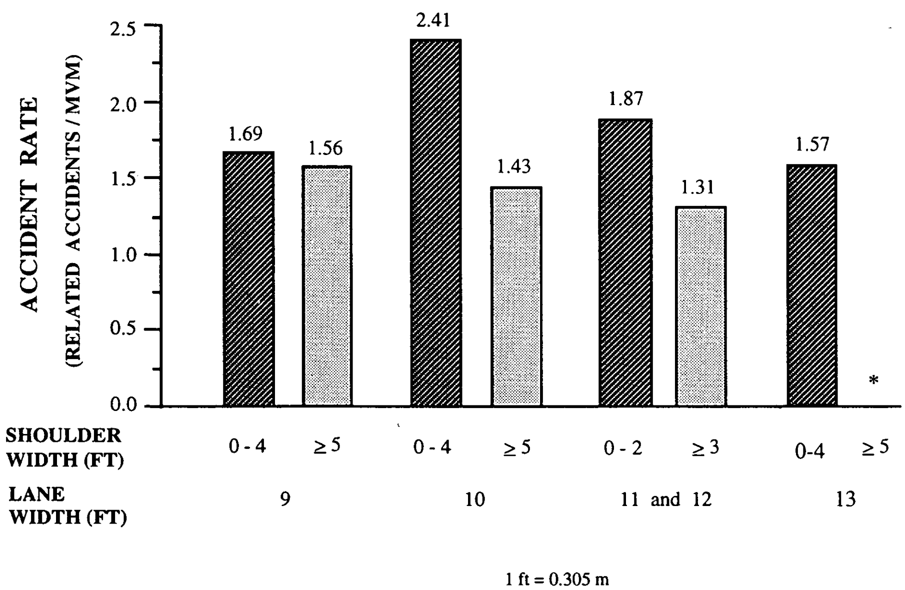

FIGURE 2 Rates of related accidents by lane and shoulder width from the data base for low-volume roads (the asterisk indicates inadequate sample size).

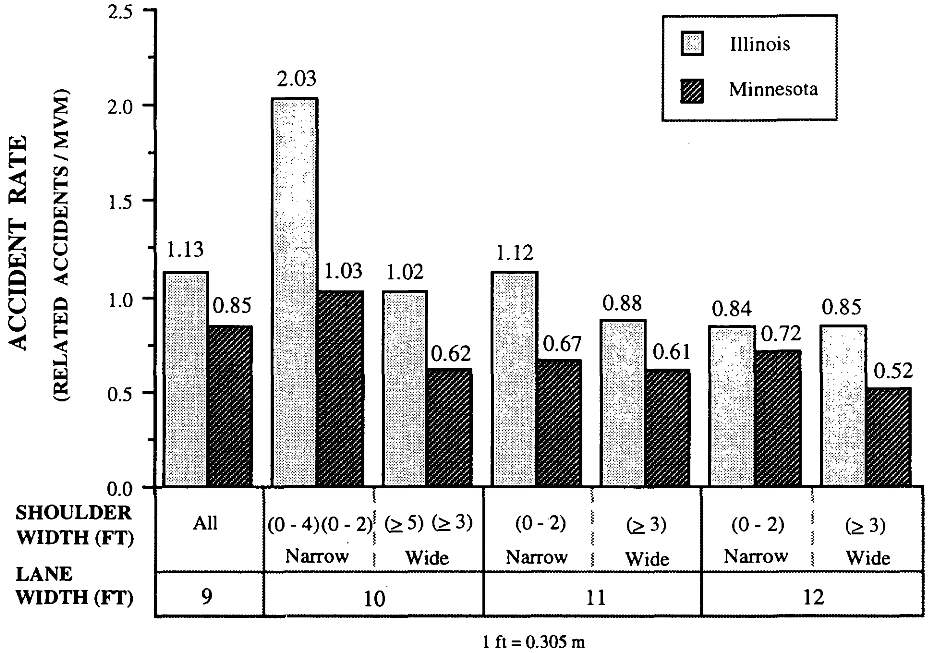

FIGURE 3 Rates of related accidents by lane and shoulder width in the Illinois and Minnesota data bases.

of the earlier analysis regarding lower accident rates for roads with 2.7-m (9-ft) lanes and higher rates for roads with 3.0-m (10-ft) lanes with narrow shoulders.

The North Carolina data showed rates of related accidents to be constant for roads with lane widths of 2.4 m (8 ft) or less and 2.7 m (9 ft), with rates of 1.95 and 1.94, respectively. In contrast to the other states and the primary data base, the rate then dropped to 1.73 for roads with 3.1-m (10-ft) lane widths and to 1.69 for roads with 3.4- and 3.7-m (11- and 12-ft) lane widths. Shoulder widths of 1.5 m (5 ft) or greater were associated with reduced accident rates. This could be due to roads with 2.7-m (9-ft) lanes in North Carolina being maintained by the state department of transportation in such a way to look like other, wider state roads (e.g., in terms of shoulder character, ditches, pavement striping), such that vehicle speeds on roads with 2.7-m (9-ft) lanes could be higher (and more likely to result in accidents) than those on roads with similar widths in other states.

It should also be mentioned that the North Carolina data supported the finding of the other data bases that increases in shoulder width reduced rates of related accidents, even though the important break points (or categories of shoulder width) varied for different lane widths and data bases. However the North Carolina data base did not show a lower accident rate for roads with 2.7-m (9-ft) lane widths than for roads with 3.1-m (10-ft) lane widths, after adjusting for shoulder width.

Discussion of Results

The results from the analysis of the primary and validation data bases have several important implications concerning safety effects of various lane and shoulder widths. First, on the basis of the data in primary data base, the presence of a wider shoulder is associated with a significant accident reduction for lane width categories of 3.0 m (10 ft) or greater. For roads with 3.1-m (10-ft) lanes, a shoulder with a width of 1.5 m (5 ft) or greater is needed to affect accident rate significantly. For roads with 3.4- and 3.7-m (11- and 12-ft) lane widths, shoulders with widths of 0.9 m (3 ft) or greater have significantly beneficial effects. For roads with lane widths of 2.7 m (9 ft), wider shoulders have a minimal, if any, safety benefit.

Second, with respect to lane width, data from two of the three validation data bases (Illinois and Minnesota) support the finding of a reduced accident rate for roads with 2.7-m (9-ft) lane widths in comparison with those for roads with 3.1-m (10-ft) lanes with narrow shoulders. Also the primary data base and the same two validation data bases both show that roads with 3.4-m (11-ft) lane widths have substantially lower accident rates in comparison with those for roads with 3.1-m (10-ft) lane widths, particularly where narrow shoulders exist. Furthermore, little if any real accident benefit can be gained from increasing lane widths from 3.4 m (11 ft) to 3.7 m (12 ft) on low-volume roads.

These analysis results generally agree with engineering intuition. Wider shoulders logically result in reduced accident rates, because drivers have more room to recover after encroaching over the edge line. Roads with lanes of 3.4 m (11 ft) or wider have lower accident rates than roads with 3.1-m (10-ft) lanes, which is again intuitively expected. The fact that 3.7-m (12-ft) lanes appear to offer minimal accident reduction in comparison with the number of accidents on 3.4-m (11-ft) lanes on low-volume roads agrees with results of a 1979 study by Zegeer et al. (*6*) of more than 16 000 km (10,000 mi) of rural, two-lane roads in Kentucky.

The main issue in question concerns the lower calculated accident rates for roadways with 2.7-m (9-ft) lanes in comparison with the accident rates for those with 3.1-m (10-ft) lanes. There are two possible explanations for this counterintuitive finding. First, the speeds on these narrower roadways may be lower, reflecting not only the effect of speed but also the effects of other variables such as functional class and terrain. The majority of roads with such narrow lanes may be more local in character, carrying lower-speed, local traffic. (Note that no speed data were collected as part of the present study.) Roadways with 3.1-m (10-ft) lanes are commonly found on higher-class facilities, on which vehicles tend to operate at higher speeds than on roads with 2.7-m (9-ft) lanes.

The analysis results support the continued use of 2.7-m (9-ft) lanes on some roadways that have lower than average accident rates, as long as these narrow roadways do not have excessively high speeds. Widening of an existing roadway with 2.7-m (9-ft) lanes to provide 3.1-m (10-ft) lanes cannot be expected to improve its safety unless such widening is accompanied by a shoulder width of at least 1.5 m (5 ft). Widening of lanes from 3.1 m (10 ft) or less (which have little or no shoulders) to 3.4 m (11 ft) plus the provision of greater than 0.6-m (2-ft) shoulders would generally be effective in terms of reducing accident rates on the basis of the results of the present analysis. The authors conclude that these findings also support construction of new roadways with 2.7-m (9-ft) lanes in certain situations (e.g., very low traffic volume, low design speeds, local traffic, and minimal truck volumes).

Issue 5: Paved Versus Unpaved Road Surface

From the primary data base rates of related accidents were compared between paved and unpaved roadway sections from states where both types of sections were available. Three different accident rate models were used to compare the safety of paved versus unpaved roads. Again each analysis controlled for important traffic and roadway variables such as state, terrain, roadside recovery distance, and roadway width. For each of three lane width categories [≤2.7, 3.0 to 3.4, 3.7 m (≤9, 10 to 11, ≥12 ft)], unpaved roads had higher rates of related accidents than paved roads. This was also true using the rate of related injury accidents.

Next a comparison between rates of related accidents for paved and unpaved roadways for various ADT categories (i.e., <250, 250 to 400, and >400 vpd) was made to determine the levels of traffic at which paved surfaces provide safety benefits. On roadways with ADT of less than 250 vpd, accident rates did not differ significantly between paved and unpaved roads. However for ADT of more than 250 vpd, rates for unpaved roads were significantly higher than those for paved roads (except for the Minnesota validation data base). Thus the results of this analysis from the primary data base provide some indication that roadways with ADT of more than 250 vpd should be paved to provide reduced numbers of accidents.

Another question concerned how total roadway width on unpaved roads affects accidents, and here the findings were in contrast to the earlier findings for paved roads. By using data for the unpaved road samples from only the primary data base, the rates of related accidents per 1.61 million vehicle km (per MVM) were much lower on roadways with total widths of less than 5.5 m (18 ft) than on roadways with total widths of 6.1 to 6.7 m (20 to 22 ft) or 7.3 m (24 ft) or greater (i.e., rates of 1.72 versus 3.95 and 3.88, respectively). Similar trends were found by using rates of accidents resulting in injuries. Thus the increased width of unpaved roadways increases accident rates, which is the reverse of the finding for paved roads. Validation data from Minnesota indicated fluctuating rates for roads with widths of 5.5 to 9.2 m (18 to 30 ft), with some decrease in rate as widths increased over 9.2 m (30 ft). Minnesota data were used for this validation because of the large sample of unpaved roadways in that state.

As with the previous discussion of roads with very narrow lane widths, speed may be an explanation for what appears to be a counterintuitive finding. Vehicles on unpaved roads that are very narrow are probably driven at very low speeds. Wider, unpaved roads may appear safer and encourage higher speeds, even though roadway alignment is severe (e.g., sharp curves), thereby increasing the potential for accidents.

In summary roads with ADT values of more than 250 vpd should in general be surfaced to improve safety. Of course those making the final decision on which unpaved roadways should be surfaced should also consider the accident experience, traffic volumes, roadway alignment (in terms of which sections can handle higher speeds safely after surfacing) on each section, as well as priorities for surfacing under available funding levels.

Furthermore the results show that the width of unpaved roads also can affect accident rates. Although accident rates fluctuate considerably for narrow roadways, accident rates for roadway widths of 6.1 m (20 ft) or less are generally low on unpaved roads. This may occur as a result of reduced vehicle speeds on very narrow, unpaved roads. As widths increase to about 9.2 m (30 ft), accident rates increase, perhaps because of increases in vehicle speeds. As widths increase further to more than 9.2 m (30 ft), rates seem to decrease again, perhaps because vehicle speeds do not increase further for unpaved roadway widths of more than 9.2 m (30 ft).

CONCLUSIONS

The major research conclusions of the present study are given below.

1. Accident rates on paved, low-volume roads are significantly reduced by wider roadway width, improved roadside condition, flatter terrain, and fewer driveways per 1.61 km (1 mi). No differences in accident rates were found on roads with paved shoulders in comparison with the rates on roads with unpaved shoulders. Accident rates are most highly correlated with lane and shoulder widths for single-vehicle and opposite-direction accidents.

2. The presence of a shoulder is associated with significant accident reductions for roads with lane widths of 3.1 m (10 ft) or greater. For roads with lane widths of 3.0 m (10 ft), shoulders of 1.5 m (5 ft) or greater are needed to reduce accident rates. For roads with lane widths of 3.4 and 3.7 m (11 and 12 ft), shoulder widths of at least 0.9 m (3 ft) result in significant accident reductions in comparison with the numbers of accidents on roads with narrower shoulders.

The study also addressed roads with lane widths of 2.7 m (9 ft) in terms of their accident experience. For a combination of reasons there is no apparent benefit in terms of reducing the number of accidents from widening such lanes from 2.7 m (9 ft) to 3.1 m (10 ft) unless shoulders of 1.5 m (5 ft) or more are also added. Indeed the study produced evidence that existing roads with 2.7-m (9-ft) lanes with narrow or wide shoulders are preferable to roads with 3.1-m (10-ft) lanes with narrow shoulders, perhaps because of lower vehicle speeds on roads with 2.7-m (9-ft) lanes and thus lower numbers of accidents.

3. Accident experience does not appear to be significantly different for unpaved versus paved roadway surfaces at traffic volumes of 250 vpd or less. At traffic volumes greater than this, accident rates are significantly greater for unpaved roadways than for paved roadways, all else being equal. Therefore paving of rural roads with traffic volumes of 250 or more vpd will generally improve their safety. Accident rates increase on unpaved roads as width increases up to 9.1 m (30 ft), perhaps because of higher vehicle speeds on wider unpaved roads.

The results of the accident data analyses were used along with other considerations in the development of recommended changes to the AASHTO guidelines for roadway widths on low-volume roads. Details of those recommended guidelines are contained in the full report of the study (*12*). It should also be mentioned that all roadway features, including roadway width, roadside features, traffic control devices, and roadway alignment, should be considered for possible improvement as needed in conjunction with resurfacing, restoration, and rehabilitation projects and for major reconstruction projects.

APPLICATION OF RESEARCH RESULTS

The research reported here was part of a larger research effort funded by NCHRP. Project 15-12, Roadway Widths for Low Traffic Volume Roads, was conducted to answer basic questions about the cost-effectiveness of design values in current AASHTO policies (*2*) for rural roads with ADT volumes of less than 2,000 vpd. Other tasks performed as part of project 15-12 included construction cost modeling, a review and synthesis of operational considerations related to roadway widths (e.g., relationship of width to operation speeds, capacity and oversize vehicle operations, and analysis of functional shoulder widths, and analysis of design value consistency within the AASHTO policy.

The final report for NCHRP 15-12 identified revisions to design values for lane width and shoulder width as a function of design speed, functional classification, terrain, and traffic volume. The draft revisions to AASHTO roadway width guidelines reflected key accident relationships reported here

1. Lane widths of 2.7 m (9 ft) may be an appropriate standard for a wider range of operating speeds and traffic volumes than is reflected in the current policy.
2. Lane width-shoulder width combinations resulting in a total dimension of 9.2 to 9.8 m (30 to 32 ft) are cost-effective for a greater range of traffic volumes than is reflected in current design policy.
3. Justification of full-width [3.7-m (12-ft)] lanes and shoulders [3.1 m (10 ft)] as a basic standard is evident only for roads with higher design speeds, roads with traffic volumes of more than 1,500 vpd, and roads with a significant proportion of heavy vehicle traffic.

The net effect of recommended changes to AASHTO policy design values would be a downsizing, particularly for highways with lower design speeds and with traffic volumes in the range of 400 to 1,000 vpd. As of the date of this paper's submission for publication, recommended revisions to the 1995 draft AASHTO policy chapter on local roads have been made. Those revisions reflect many of the research findings. Revisions to design values in the collectors and arterials chapters are also expected.

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Effect of Urban and Suburban Median Types on Both Vehicular and Pedestrian Safety

BRIAN L. BOWMAN AND ROBERT L. VECELLIO

Urban and suburban traffic engineers have the difficult task of providing safe roadway cross sections for all types of users. The major difficulty arises at most central business districts and at many suburban locations because of relatively high volumes of pedestrians attempting to cross roadways with minimum available traffic gaps. This is due to differences in operating characteristics between vehicles and pedestrians. Measures to increase vehicular capacity on arterials often result in potential hazards to pedestrians. It has long been recognized that medians are an effective method of increasing vehicular safety and capacity on urban and suburban arterials. Also medians are generally considered to be beneficial for pedestrian safety and operations, but their actual effects are unknown. The results of a study sponsored by FHWA on the impact of median types on the safety of vehicles and pedestrians are presented. The study included the analysis of 32,894 vehicular and 1,012 pedestrian accidents occurring in three cities on arterials with the following median types: (a) raised, (b) flush or two-way left-turn, and (c) no existing median (undivided).

Urban traffic engineers have the difficult task of providing safe and efficient traffic facilities for all roadway users. These traffic facilities must be provided at locations that often have both high vehicular and high pedestrian volumes. The difficulty arises because of the conflicting needs of motorists and pedestrians and their respective behavioral characteristics.

As an example consider the concerns that must be addressed during the selection of an optimal traffic signal timing plan. The duration of the green phase must provide sufficient time for pedestrians to cross and simultaneously to satisfy vehicular needs. In this instance the needs of pedestrians and vehicular traffic can be in conflict with each other. This often occurs at the intersection of a high-volume roadway with a relatively low-volume minor roadway. In addition the major street is often wide and the proportionate green time to the minor street is small. The pedestrians are, however, crossing the wide major roadway with the minor street green time. The selection of minimum green time on the basis of slower walking speeds in these instances can result in increased delay to the major arterial traffic.

Problems at signalized intersections are also complicated by geometric design and vehicle movement paths. Unless lead left signal phasing is used, the majority of vehicular left turn movements take place at the end of the through green phase. At this time slower-moving pedestrians may still be in the crosswalk, partially fatigued, and concerned with arriving at the far curb line. Conversely the left-turning motorist is primarily concerned with oncoming traffic and may not be aware of pedestrians in the crosswalk. The result is an increased potential for pedestrian and vehicular conflicts and subsequent accidents. Solutions to the problems include separating the paths of pedestrians and vehicles, narrowing the roadway cross section at intersections, and providing medians.

Medians are classifications of traffic control islands defined as areas between traffic lanes that separate opposing traffic flows. Medians can be designed to serve more than one purpose, including controlling or providing space for vehicular crossover or other turning movements, providing a landscape area, channelizing traffic, and providing pedestrian refuge. It has long been recognized that the use of medians is an effective method of increasing vehicular safety and capacity on urban and suburban arterials. Medians help to provide more efficient use of through traffic lanes by removing left-turning vehicles from the traffic stream. Also medians generally can be considered to be beneficial for pedestrian safety and operations, but their actual effects are unknown.

This paper presents the accident analysis results of a project sponsored by FHWA. The objective of the analysis was to determine the impact of median installation on the safety of both motorists and pedestrians. The study included medians constructed on urban and suburban arterials with unrestricted land use adjacent to the street.

DATA COLLECTION METHODOLOGY

A literature review and state-of-the-practice survey conducted as part of the project indicated that raised-curb medians and two-way left-turn (TWLT) median lanes are the predominant median types currently being installed. The efforts of the project were therefore concentrated on arterials with raised-curb medians, TWLT lane medians, and undivided (no existing median) arterials. The undivided arterials are included to provide the base or control data by which to measure the safety effects of sites with medians.

Fifteen arterial sites, each with a raised curb, TWLT median, and undivided cross sections, were randomly selected from the cities of Atlanta, Georgia; Phoenix, Arizona; and Los Angeles and Pasadena, California. Each selected arterial was inspected to verify a homogeneous roadway and median design and to establish, with the local officials, that significant changes had not occurred at the location over the accident analysis time period. These determinations resulted in final site selections and the requests for accident data.

Each selected arterial was videotaped over its entire length to obtain roadway and land use characteristics. The videotapes for each arterial were reviewed, and those variables identified from prior studies as being relevant and as potentially relevant for the

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present study were extracted. Each median type was divided into midblock segments, with a segment defined as a roadway link subtended by signalized intersections. If this process resulted in segments that were shorter than 0.16 km (0.1 mi) long, they were excluded from the midblock analysis. The variables were identified and extracted separately for midblock segments and signalized intersections. This method of extraction permitted the flexibility of analyzing midblock segments, signalized intersections, and the entire length of arterial (i.e., segments and signalized intersections) separately.

A total of 33,139 vehicle and 1,328 pedestrian accidents were initially identifed as being relevant to the study. A detailed printout of each pedestrian accident that occurred on the major road and within 46 m (150 ft) of the major road curb line was requested. Reports on all pedestrian accidents within 46 m (150 ft) of the major roadway were requested to help ensure that all relevant accidents were included. Without this type of request, accidents involving a pedestrian crossing the minor roadway (i.e., traveling along the arterial being analyzed) and being struck by a vehicle turning left from the major roadway may have been lost. The possible effects of medians on the behavior and visual search patterns of left-turning motorists would therefore have been neglected. Copies of the original pedestrian accident reports were obtained, and the verbal descriptions and accident diagrams were reviewed to determine whether the project criteria were satisfied. The scenario used in the extraction and coding of the accident data base is summarized below.

- Pedestrian accidents that occurred on the arterial, including vehicles from the minor road, and within 30 m (100 ft) of the arterial centerline and involving a major arterial vehicle were included in the analysis. The original request for reports of pedestrian accidents within 30 m (150 ft) was made to enable an inspection of the verbal accident description and the subsequent identification of erroneously located accidents.
- Vehicular accidents on the arterial segments [i.e., arterials subtended by signalized intersections 0.2 km (0.1 mi) or longer], and for signalized intersections considered separately, include only accidents that occurred on the arterial. Vehicular accidents on the minor roadway were not included.
- Traffic volume counts were obtained for each arterial. When available, these counts were summarized for each analysis year. In many instances annual counts were not available, necessitating the use of growth factors to increase or decrease the traffic volumes as appropriate.
- Pedestrian traffic accidents involve the presence of both a vehicle and a pedestrian in the same place at the same time. Estimates of vehicular presence are available to most agencies in the form of average daily traffic (ADT) counts. Pedestrian volumes are not available to most agencies, resulting in the need to develop surrogate measures of pedestrian activity. The intensity of pedestrian presence is inherently assumed to be represented by the area [i.e., central business district (CBD) or suburban area] and the type of land use.

DATA ANALYSIS

Physical Data

The data collection sheets completed during the field review of each site, the videotapes, and city maps were used to extract the physical data for each site. The data were organized into a data base that contained a city and segment identifier that permitted a merge with the accident data base.

The majority of arterial miles (84.7 percent) included in the study was found in suburban areas. This disparity is due to the relatively limited number of miles available in CBD areas compared with the large number of suburban miles within the CBD of each city. The limited size, development intensity, and city traffic engineering practices resulted in an inability to identify each median type of sufficient length for study purposes. Control over site selection was exercised to ensure that the combined arterial miles for each median type were sufficient for CBD and suburban data reliability. This was accomplished by increasing the analysis miles of deficient median types in some cities and by increasing the years of pedestrian accident analysis. The largest number of arterial miles were obtained from the Atlanta area. A total of 234.8 km (145.9 arterial mi) was analyzed for the project.

Land use was classified as residential, office, and business since land use type was suspected to influence the type and quantity of pedestrian activity. The original land use classifications included subcategories on land use type such as single-dwelling residential, high-rise residential, shopping center, and strip commercial. This subdivision of land use, however, resulted in large data stratifications that provided few useful data. Residential, therefore, indicates land use that varies from single dwellings to multistory apartment structures. Office refers to land use that does not entail large movements of customers during the business day and where employees are the primary trip makers. The commercial designation includes business activity that depends on customer visitation to the establishment. The predominant land use in the vicinity of the suburban arterials was residential, whereas only 0.5 km (0.3 mi) of CBD area was residential.

It should be noted that the land use varied drastically not only along the arterial length but also on each side of the roadway. To compensate for this variation the predominant land use was coded for each segment of the arterial (i.e., between signalized intersections). There were many instances, however, in which residential use would occupy one side of the arterial and another use such as commercial would occupy the opposite side. In these instances the land use was assigned on the basis of judgment related to the observed activity at the time of the field survey.

Minor variations in volumes were experienced at some sites because of annual volume counts and growth factors. The volumes for the majority of sites were within the range of 20,000 to 50,000 per day and remained relatively consistent during the analysis period. Since the project concentrated on CBD and subruban sites, the volumes of all the studied arterials were relatively high. Only a limited number of arterials had volumes of fewer than 15,000 vehicles per day.

Table 1 gives the number of arterial miles by traffic lane cross section for each median type. All of the raised-curb and TWLT median sites consisted of four to six through lanes. The flush center lane for TWLT arterials was not counted as a traffic lane. In the category of undivided arterials, 8.2 km (5.1 mi) of an odd number of lanes is included. These roadways consist of an unbalanced number of lanes to facilitate traffic movement during the peak hours (e.g., three lanes in one direction and two lanes in the other direction). The flush center lane is then used for reverse traffic flows during the other daily peak hour. These special cross sections are located in Phoenix.

TABLE 1 Summary of Arterial Miles by Number of Traffic Lanes

	Miles of Arterials by Number of Through Lanes and Median Type										
	Raised			TWLT			Undivided				
Thru Lanes	4	5[1]	6	4	5[1]	6	2	3[1]	4	5[1]	6
CBD	0.3	--	29.7	5.7	1.2	0.9	--	0.6	6.1	--	1.7
Suburban	13.0	3.1	5.8	26.9	11.6	8.8	2.9	--	22.1	4.5	1.0

1 mi = 1.6 km
[1]Unbalanced number of lanes to facilitate peak hour traffic flow

Accident Data

The accident data provided for each median arterial were coded into a uniform format and merged with the physical features data base. Separate data bases were maintained for vehicular and pedestrian accidents. Vehicular accidents included only those accidents that were coded as occurring on the arterial. The pedestrian data base included accidents coded as occurring on the arterial and on the minor roadway, within 30 m (100 ft) of the median arterial centerline, that involved an arterial vehicle.

Summary rates for the arterials were determined by summing the accident frequency and dividing that number by the sum of the annual volume (ADT_i) and median length (L_i) product for each median segment. The equation can be written as

$$\left(\sum_{i=1}^{n} \text{Annual accident frequency for segment } i\right) \Big/ \left(365 \sum_{i=1}^{n} \mathrm{ADT}_i L_i\right)$$

where n = number of segments. This provides a weighted average estimate that is better than the estimate obtained by averaging the rates of each individual section for comparing rates for arterials. The rates were then multiplied by 100 million vehicles to obtain a sufficient number of significant figures for analysis of pedestrian accidents.

The accident data were tested to determine whether there were significant differences between the accident rates of selected data sets. Statistical tests were performed by using each categorized site and its respective accident rate in the data base. Because the data had been converted to rates and a large number of observations existed for each data set, the data were considered as being normally distributed between analysis groups. Student's t-test was used to determine whether a statistical difference existed between two data sets. The procedure was applied by using the SAS computer statistical analysis package (*1*). The first step in the application of the t-test was to develop an F-statistic to test for equality of the variances. This was necessary because the SAS procedure computes two t-statistics. One is based on the assumption that the variances of the two sets are equal, and the other is based on unequal variances.

Comparisons between more than two groups at a time were performed by simultaneously comparing the variability of group means about the overall mean (between estimate) relative to the variability of each observation to its respective group mean (within estimate). This procedure, known as analysis of variance, (ANOVA), established when sufficiently large differences existed between the groups to determine that a significant difference existed. The Scheffé multiple comparison test was then used to establish simultaneous confidence intervals between all possible combinations of group pairs. The means of the paired groups were considered unequal, and therefore significantly different, when the confidence interval did not contain zero. All statistical tests were conducted as two-tailed 95 percent confidence interval tests.

An initial premise of the study was that pedestrian activity and intensity in CBD areas were different from those in suburban areas. If this premise were true then different models for each area, or the inclusion of area as an independent variable, would provide enhanced accident prediction capabilities. Table 2 contains the result of the t-test performed to determine whether there is a significant difference in the accident rate between CBD and suburban areas for the different median types. Pedestrian accidents occurred

TABLE 2 Significant Difference and Mean Accident Rates Between CBD and Suburban Areas

Ho: CBD accident rate = suburban accident rate. Significance level (of t - test) = 0.05.									
	Raised			TWLT			Undivided		
		Mean Rate[1]			Mean Rate[1]			Mean Rate[1]	
Accident Category	α	CBD	Sub	α	CBD	Sub	α	CBD	Sub
vehicle	0.2463	471.64	384.02	0.0320*	475.17	611.27	0.0798	835.85	627.68
pedestrian	0.0304*	26.30	9.23	0.0027*	31.55	13.11	0.000*	95.42	28.28

*Denotes significant difference.
[1]Summary - accident rates expressed in accidents per 10^8 vehicle miles
1 mi = 1.6 km

TABLE 3 Accident Frequency and Associated Rate for Arterials

Area	Accident Category	Raised	TWLT	Undivided
CBD	Veh	1663	2019	2509
	Rate[1]	623.06	513.79	905.21
	Ped	51	162	242
	Rate[1]	19.11	41.11	87.31
SUB	Veh	7535	14828	4340
	Rate[1]	373.00	676.29	409.22
	Ped	128	282	147
	Rate[1]	6.31	12.89	13.91

[1]Arterial accident rates in 10^8 vehicle miles
1 mi = 1.6 km

at a significantly higher rate in CBD areas for all of the median types. Vehicular accidents did not exhibit as large an influence by area as did pedestrian accidents. The impact of raised medians on vehicular accidents is significantly different between CBD and suburban areas, with the average rate of vehicular accidents being higher in CBD areas.

The frequency of vehicular and pedestrian accidents occurring on the arterials and the associated summary rates are provided in Table 3. In the CBD the accident rate for undivided arterials is higher for both vehicles and pedestrians than that for arterials with raised and TWLT medians in the same type of area. Although this is not unexpected, it is also interesting to note that the vehicular accident rates in CBD areas are higher on arterials with raised-curb medians than on those with TWLT medians. In both CBD and suburban areas arterials with raised-curb medians displayed a lower pedestrian accident rate than the arterials with either a TWLT lane or no median.

The accident rates for the different median types within CBD or suburban areas were analyzed to determine whether the differences in the accident rates shown in Table 3 were sufficiently large to be significant. Statistical significance was determined by performing ANOVA on the accident rate of each categorized site for the arterials. The results of the statistical test, presented in Table 4, indicate that there were significant differences in the accident rates between arterials with raised and TWLT median types and undivided cross-sections in both the CBD and suburban areas.

TABLE 4 Statistical Difference Between Accident Rates[1] on Arterials with Raised-Curb and TWLT Medians and Undivided Cross Sections for CBD and Suburban Areas

Ho: accident rate of raised = accident rate of TWLT = accident rate of undivided
Significance level (critical ∝) = 0.05

	Source[2]	DF	Mean Square	F	Prob. > F	Significant
CBD						
vehicle	Between	2	1928888			
	Within	115	293918	6.56	0.0020	yes
pedestrian	Between	2	63141			
	Within	115	4508	14.01	0.0001	yes
SUBURBAN						
vehicle	Between	2	2162291			
	Within	323	204462	10.58	0.0001	yes
pedestrian	Between	2	9990			
	Within	323	2062	4.85	0.0084	yes

[1]Accident rates expressed in accidents per 10^8 vehicle miles
[2]Between is the variability of vehicle and pedestrian group means to overall mean.
Within is the variability of vehicle and pedestrian observations to their group mean.
1 mi = 1.6 km

When differences existed they were determined to be significant or not by performing the Scheffé multiple comparison test, the results of which are presented in Table 5. For vehicular accidents there was a significant difference between those on arterials with a TWLT median and those on arterials with an undivided cross section in CBD areas, and in suburban areas, there was a significant difference between accidents on arterials with a raised median, a TWLT and raised median, and an undivided cross section.

Therefore in CBD areas arterials with undivided cross sections have significantly higher vehicular accident rates than do those with TWLT medians. In suburban areas arterials with raised medians have a significantly lower vehicular accident rate than do those with TWLT medians and undivided cross sections. In addition there are significantly fewer pedestrian accidents on arterials with raised medians than on those with undivided arterials in suburban areas, and there are fewer pedestrian accidents in CBD areas for arterials with both TWLT medians and undivided cross sections.

Because of the different operational characteristics and the effects of the median on safety, the data in Table 3 were divided into midblock segment and signalized intersection accidents and are presented in Table 6. The median arterial accidents shown in Table 3 are less than the total number of accidents described in Table 6 because of data verification and editing. In a limited number of cases the median type changed for a short length of roadway between signalized arterials or sufficient data could not be reliably extracted from the videotapes because vehicles blocked the visual field or because land uses were too few for statistical reliability (e.g., industrial). In these instances the median segments were dropped from the analysis, but the intersection data were retained for use in the analysis of isolated intersections. The arterial accident data in Table 3 include the midblock and signalized intersection accidents that are appropriate for analyzing arterial lengths with specific median types. The data on midblock segment accidents in Table 6 include data for all accidents that did not occur within 30 m (100 ft) of the crossroad intersection centerline. The rates for midblock segments are based on the ADT of the arterial. The rates for the signalized intersections are based on the total number of entering vehicles.

The CBD vehicular accident rates, presented in Table 6, for arterials with raised medians, both midblock and signalized intersection locations, were higher than those for arterials with TWLT medians and undivided cross sections. This difference is very pronounced at CBD signalized intersections with raised-curb medians. The rate is more than 3 times greater than that for arterials with undivided cross-sections and almost 13 times greater than that for arterials with a TWLT median design. This disparity can be explained by considering that curbed medians concentrate left-turn maneuvers at median cross-over points and major intersections. Therefore on short median segments, as would occur within CBD areas, vehicle turning movements would be concentrated at the signalized intersections. The pedestrian accident rate for CBD areas is lower at midblock locations with raised medians than at TWLT medians or undivided cross sections.

TABLE 5 Scheffé Multiple Comparison Tests Between Median Types for Vehicular and Pedestrian Accident Rates on Arterials

VEHICLE ACCIDENTS			
	95% Simultaneous Confidence Interval		Significant[1]
CBD			
raised, TWLT	-388.7	381.6	no
raised, undivided	-735.0	6.6	no
TWLT, undivided	-631.2	-901.0	yes
SUBURBAN			
raised, TWLT	-371.8	-82.7	yes
raised, undivided	-397.5	-89.9	yes
TWLT, undivided	-176.6	143.8	no
PEDESTRIAN ACCIDENTS			
CBD			
raised, TWLT	-52.9	42.5	no
raised, undivided	-115.0	-23.2	yes
TWLT, undivided	-97.4	-30.4	yes
SUBURBAN			
raised, TWLT	-18.4	10.6	no
raised, undivided	-34.5	-3.6	yes
TWLT, undivided	-31.3	0.9	no

[1]Pairs are significantly different if confidence interval does not contain zero.

TABLE 6 Accident Frequency and Associated Rate for Midblock Median Segments and Signalized Intersections

Area	Location	Accident Category	Raised	TWLT	Undivided	Freq Totals
CBD	midblock	vehicle				
		freq	558	626	564	1748
		rate[1]	209.06	159.34	203.48	--
		pedestrian				
		freq	26	46	60	132
		rate[1]	9.74	11.71	21.65	--
	signalized inter	vehicle				
		freq[1]	1105	137	34.7	4659
		rate[2]	144.96	11.23	45.91	--
		pedestrian				
		freq	25	16	299	340
		rate[2]	3.28	1.31	4.02	--
Suburban	midblock	vehicle				
		freq	3823	6827	2241	12891
		rate[1]	189.23	311.37	211.31	--
		pedestrian				
		freq[1]	78	146	71	295
		rate	3.86	6.66	6.69	--
	signalized inter	vehicle				
		freq	4229	7507	2105	13841
		rate[2]	87.43	136.36	68.79	--
		pedestrian				
		freq[2]	47	137	71	255
		rate	0.97	2.49	2.32	--
Frequency			9,715	15,097	8,327	33,139
TOTAL vehicles, pedestrians			176	345	501	1,022

[1]Midblock accident rate per 10^8 vehicle miles
[2]Intersection accident rate per 10^8 entering vehicles
1 mi = 1.6 km

Table 7 gives the predominant vehicular accident types for CBD and suburban areas that occur on arterials with raised or TWLT medians and undivided roadways. The vehicular accident rates exhibited were unexpected, and at first they were believed to be erroneous. This resulted in a reverification of the data base. For example the rear-end accident rate on arterials with raised medians in CBD areas is higher than that on arterials with TWLT medians and undivided cross sections. The reason for this is not known with certainty, since the total miles of arterials with raised TWLT medians and undivided cross sections included in the analysis with the CBD areas were approximately equal. One possible explanation is that left turns are often prohibited from undivided roadways at midblock locations within the CBD, thereby reducing the potential for rear-end accidents. In the majority of cases the accident rates in CBD areas are less than those in suburban areas.

The determination of statistical significance of vehicular accident types between midblock median types for both CBD and suburban areas is presented in Table 8. A significant difference in rear-end, right-angle, and left-turn vehicular accident rates between the different median types was exhibited in suburban areas. This significant difference was also exhibited with right-angle accident types in CBD areas. The median types that exhibited the difference, as determined by the Scheffé multiple comparison test, are given in Table 9. Raised medians have a significantly lower midblock accident rate when comparing arterials with raised medians with those with TWLT medians for rear-end, right-angle,

TABLE 7 Summary of Predominant Midblock Vehicle-to-Vehicle Accident Types

Accident Type	Raised		TWLT		Undivided		Freq TOTAL
	CBD	Suburb	CBD	Suburb	CBD	Sub	
Rear End							
freq	269	1636	172	3061	179	1007	6324
rate[1]	100.78	80.98	43.78	139.61	64.58	94.95	--
Right Angle							
freq	70	708	73	1387	94	405	2737
rate[1]	26.23	35.05	18.58	63.26	33.91	38.19	--
Head-On							
freq	2	27	14	56	9	22	130
rate[1]	0.75	1.34	3.56	2.55	3.25	2.07	--
Left Turn							
freq	57	492	86	1151	53	232	2071
rate[1]	21.36	24.35	21.89	52.50	19.12	21.88	--
Other							
freq	160	960	281	1172	229	575	3377
rate[1]	59.95	47.52	71.53	53.45	82.62	54.22	--
FREQ TOTAL	558	3823	626	6827	564	2241	14639

[1]Accident rates expressed as accidents per 10^8 vehicle miles
1 mi = 1.6 km

and left-turn accident types and also when comparing arterials with raised medians with those with undivided medians for right-angle accident types in suburban areas. Arterials with TWLT medians have significantly higher rear-end and left-turn accident rates than undivided arterials at suburban midblock segments. There were 29 CBD and suburban head-on type accidents at midblock segments with raised medians. These accidents were analyzed to determine what driver actions contributed to the accidents and the associated severity. Ten of the 29 head-on accidents (34.5 percent) were the result of motorists traveling the wrong way and 3 (10.3 percent) occurred in the median crossover resulting from a left-turning maneuver. The majority of the raised medians where head-on accidents occurred (82.8 percent) had a width of 2.4 m (8 ft) or more. The severity rates for head-on accidents are summarized in Table 10 and indicate that midblock, head-on types of accidents at raised medians are less severe than those on arterials with TWLT medians.

A summary of accident severity by median type for midblock segments is presented in Table 11. A greater percentage of accidents occurred at raised medians but were of lower severity [property damage only (PDO)] than at TWLT medians and undivided cross-sections. The severity of accidents on arterials with TWLT medians is greater in CBD areas but is less in suburban areas in comparison with the severity of accidents on undivided arterials. Tests on the accident severity rates presented in Table 12 indicate the statistical difference between the median types for both CBD and suburban areas for accidents involving property damage only and personal injury. The lack of significant differences in the fatality rates is due to relatively small sample sizes. The multiple comparison tests summarized in Table 13 indicate that arterials with raised medians in both CBD and suburban areas have significantly lower personal injury rates than arterials with TWLT medians.

Findings and Conclusions

- It was initially assumed that pedestrian activity, and hence pedestrian accident rate, is higher in CBD areas than in suburban areas. This assumption was necessary since actual pedestrian volumes for roadway segments were not available. The assumption was tested by developing pedestrian accident rates on the basis of pedestrian accident frequency, vehicular volumes, and roadway length. Pedestrian accidents occurred at a significantly higher rate in CBD areas than in suburban areas for all three median types. CBD and suburban areas can, therefore, be used as surrogate measures of pedestrian activity. In addition the development of models to predict pedestrian accidents should be performed separately for CBD and suburban areas.
- Vehicular accidents do not exhibit as large an influence by area as do pedestrian accidents. This supports the assumption that CBD areas have more pedestrian activity than suburban areas.
- Arterials with TWLT medians located in CBD areas had lower vehicular accident rates than those with raised-curb medians and undivided cross sections. Undivided arterials had the highest vehicular accident rates in CBD areas. Comparisons between the

TABLE 8 Statistical Difference in Midblock Vehicular Accident Types Between Median Types for CBD and Suburban Areas[1]

Ho: accident type rate of raised = accident type rate of TWLT = accident type rate of undivided
Significance level (critical α) = 0.05

	Source	DF	Mean Square	F	Prob. > F	Significant
REAR END						
CBD	Between	2	12218	1.00	0.3681	no
	Within	578	12205			
SUBURBAN	Between	2	503609	28.39	0.0001	yes
	Within	1222	17739			
RIGHT ANGLE						
CBD	Between	2	13544	3.33	0.0365	yes
	Within	578	4067			
SUBURBAN	Between	2	104307	12.20	0.0001	yes
	Within	1222	8548			
HEAD ON						
CBD	Between	2	195	1.57	0.2084	no
	Within	578	124			
SUBURBAN	Between	2	46	0.42	0.6548	no
	Within	1222	109			
LEFT TURN						
CBD	Between	2	759	0.20	0.8168	no
	Within	578	3753			
SUBURBAN	Between	2	144226	33.24	0.0001	yes
	Within	1222	4339			

[1]Accident rates expressed as accidents per 10^8 vehicle miles
1 mi = 1.6 km
[2]Between is the variability of CBD and suburban group means to overall mean
Within is the variability of CBD and suburban observations to their group mean

three types of cross sections revealed that arterials with TWLT medians had significantly lower vehicular accident rates than undivided arterials. No significant differences were identified between comparisons of vehicular accident rates between arterials with raised-curb and TWLT medians or raised-curb medians and undivided cross sections.

- Pedestrian accident rate for CBD locations and undivided arterials was significantly higher than those for arterials with raised-curb and TWLT medians. The pedestrian accident rate for arterials with raised-curb medians was lower than those for arterials with TWLT medians and undivided cross sections in CBD locations.
- In suburban areas arterials with raised-curb medians had significantly lower vehicular accident rates than arterials with TWLT medians and undivided cross sections.
- Arterials with raised-curb medians in suburban areas had the lowest pedestrian accident rate. Arterials with raised-curb medians had a significantly lower pedestrian accident rate than arterials with undivided cross sections. There was not a significant difference between the pedestrian accident rates on arterials with raised-curb and TWLT medians.
- In suburban areas arterials with raised-curb medians had significantly lower vehicular accident rates than arterials with TWLT medians for rear-end, right-angle, and left-turn accident types. Arterials with raised-curb medians also had significantly lower accident rates than arterials with undivided cross sections for right-angle-type vehicle accidents.
- In both CBD and suburban locations arterials with raised-curb medians had lower vehicular accident rates with personal injuries than arterials with TWLT medians and undivided cross sections. The vehicular accident rates with personal injuries on arterials with raised-curb medians were significantly lower than those on arterials with TWLT medians in suburban areas. Undivided arterials had lower vehicle personal injury rates than arterials with TWLT medians in suburban areas.
- Study results indicate that, when possible, arterials with undivided cross sections should not be used in CBD areas. In CBD areas undivided arterials result in the highest accident rates for both pedestrians and vehicles.
- With one exception there is no significant difference in either pedestrian or vehicular accident rates between arterials with

TABLE 9 Scheffé Multiple Comparison Tests of Midblock Vehicular Accident Type Between Median Types in CBD and Suburban Areas

		95% Simultaneous Confidence	Interval	Significant[1]
REAR END				
SUBURBAN	raised,TWLT	-81.6	-38.1	yes
	raised, undivided	-24.2	23.3	no
	TWLT, undivided	35.4	83.2	yes
RIGHT ANGLE				
CBD	raised, TWLT	-16.54	21.83	no
	raised, undivided	-30.00	6.43	no
	TWLT, undivided	-28.92	0.06	no
SUBURBAN	raised, TWLT	-44.3	-14.2	yes
	raised, undivided	-39.0	-6.1	yes
	TWLT, undivided	-9.9	23.3	no
LEFT TURN				
SUBURBAN	raised, TWLT	-41.8	-20.3	yes
	raised, undivided	-9.8	13.7	no
	TWLT, undivided	21.2	44.8	yes

[1]Pairs are significantly different if confidence interval does not contain zero.

TABLE 10 Summary of Head-on Vehicular Accident Rates for Midblock Segments by Median Type and Area

Severity	Raised		TWLT		Undivided	
	CBD	Suburban	CBD	Suburban	CBD	Suburban
PDO						
frequency	2	12	4	26	0	5
rate[1]	0.75	0.59	1.02	1.19	0	0.47
Injury						
frequency	0	15	10	28	9	16
rate[1]	0	0.74	2.55	1.28	3.25	1.51
Fatal						
frequency	0	0	0	2	0	1
rate[1]	0	0	0	0.09	0	0.09

[1]Midblock segment rates in accidents per 10^8 vehicle miles.
1 mi = 1.6 km

TABLE 11 Summary of Midblock Vehicular Accident Severity by Median Type[1]

Severity	Raised		TWLT		Undivided	
	CBD	Suburban	CBD	Suburban	CBD	Suburban
PDO						
frequency	401	2649	266	4855	3.42	1451
rate	150.24	131.12	67.71	221.43	123.39	136.82
percent	71.9	69.3	42.5	71.1	60.6	64.8
Injury						
frequency	156	1169	360	1962	222	783
rate	58.45	57.86	91.63	89.48	80.09	78.83
percent	28.0	30.6	57.5	28.7	39.4	34.9
Fatal						
frequency	1	5	0	10	0	7
rate	0.37	0.25	0	0.46	0	0.66
percent	0.1	0.1		0.2	0	0.3

[1]Midblock segment accident rate in accidents per 10^8 vehicle miles.

TABLE 12 Statistical Difference in Midblock Vehicular Accident Severity Rates Between Median Types for CBD and Suburban Areas[1]

Ho: accident severity of raised = accident severity of TWLT = accident severity rate of undivided
Significance level = 0.05

	Source	DF	Mean Square	F	Prob. > F	Significant[2]
PROPERTY DAMAGE ONLY						
CBD	Between	2	125344	4.08	0.0174	yes
	Within	578	30743			
SUBURBAN	Between	2	1146617	24.79	0.0001	yes
	Within	1222	46260			
PERSONAL INJURY						
CBD	Between	2	73224	4.47	0.0119	yes
	Within	578	16399			
SUBURBAN	Between	2	85213	9.66	0.0001	yes
	Within	1222	8817			
FATAL						
CBD	Between	2	2.04	2.42	0.0901	no
	Within	578	0.84			
SUBURBAN	Between	2	1.51	0.11	0.8968	no
	Within	1222	13.83			

[1]Accident rates in accidents per 10^8 vehicle miles
[2]Between is the variability of CBD and suburban group means to overall mean
Within is the variability of CBD and suburban observations to their group mean
1 mi = 1.6 km

TABLE 13 Scheffé Multiple Comparison Tests of Accident Severity Between Median Types for Midblock Locations

VEHICLE ACCIDENTS				
		95% Simultaneous Confidence	Interval	Significant[1]
PROPERTY DAMAGE ONLY				
CBD	raised, TWLT	-16.5	88.9	no
	raised, undivided	-59.5	40.6	no
	TWLT, undivided	-85.5	-5.8	yes
SUBURBAN	raised, TWLT	-134.6	-64.4	yes
	raised, undivided	-71.8	4.9	no
	TWLT, undivided	27.5	104.7	yes
PERSONAL INJURY				
CBD	raised, TWLT	-85.3	-8.3	yes
	raised, undivided	-65.8	7.3	no
	TWLT, undivided	-11.5	46.7	no
SUBURBAN	raised, TWLT	-39.7	-9.1	yes
	raised, undivided	-41.1	-7.7	yes
	TWLT, undivided	-16.9	16.8	no

[1]Pairs are significantly different if confidence interval does not contain zero.

raised-curb and TWLT medians. The one exception was vehicular accident rates in suburban areas, where accident rates on arterials with raised-curb medians were significantly less than those on arterials with TWLT medians.

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Assessment of Current Practice in Selection and Design of Urban Medians To Benefit Pedestrians

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It is known that medians are an effective method of increasing vehicular safety and capacity on urban and suburban arterials. Medians can provide an additional lane for through traffic by removing left-turning vehicles from the traffic stream. Medians are also considered to be beneficial to pedestrian safety and operations, but their actual effects are unknown. The partial results of a study sponsored by FHWA are presented. A literature search and a state-of-the-practice survey regarding the effectiveness of alternative median designs and the availability of warrants, guidelines, and criteria for median installations in counties, cities, and states are summarized. The impact of median design on pedestrian safety is emphasized. The results are applicable to urban and suburban locations.

Medians and refuge islands are classifications of traffic control islands defined as areas between traffic lanes for control of vehicle movements or for pedestrian refuge. Medians can be designed to serve more than one purpose, including controlling or protecting vehicle crossover or other turning movements, providing a landscaped area, channelizing traffic, and providing pedestrian protection. Pedestrian refuge islands are specifically designed to provide a place of safety for pedestrians who cannot safely cross the entire roadway width at one time because of changing traffic signals or oncoming traffic.

Refuge islands are particularly useful at locations where heavy volumes of vehicular traffic make it difficult and dangerous for pedestrians to cross the roadway (*1*). The Manual on Uniform Traffic Control Devices (*2*) states that refuge islands are particularly useful (a) on multilane roadways, (b) in large or irregularly shaped intersections, and (c) at signalized intersections to provide a place of safety between different traffic streams.

It has long been recognized that medians are an effective method of increasing vehicular safety and capacity on urban and suburban arterials. Medians can provide an additional lane for through traffic by removing left-turning vehicles from the traffic stream. Medians are also generally considered to be beneficial to pedestrian safety and operations, but their actual effects are unknown.

This paper presents the results of an extensive literature search and state-of-the-practice survey that was conducted as part of a study sponsored by FHWA. The relevant objectives were (a) to conduct a literature review of the impacts of medians, emphasizing research of cases in which pedestrian safety was an issue and (b) to conduct a state-of-the-practice survey of state, county, and city agencies regarding current warrants, guidelines, and criteria for median installation.

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Nearly 70 articles were reviewed in the literature search, with about 50 percent of those published since 1982, reflecting renewed interest in this subject. The literature focus was on studies performed in urban and suburban locations, since rural locations do not have significant amounts of pedestrian traffic. The literature was used to identify existing guidelines that can be used to determine the appropriate median treatment to use.

MEDIAN TYPES

Raised medians promote safety and through traffic service by preventing left turns and U-turns across the medians except at designated crossover points. In addition to preventing left turns, raised medians reduce friction in the traffic stream by separating opposing traffic. The term *raised median* used herein implies the use of a curb. The effectiveness and utility of the median increase with increased width. If the raised median is at least 1.2 m (4 ft) wide it may be used by pedestrians as a rest area, enabling them to cross only one direction of traffic at a time. However, a 1.8-m (6-ft) median width is needed to accommodate multiple pedestrians in urban settings, persons with baby strollers, and wheelchairs propelled by attendants. If the median width is at least 3.0 m (10 ft) it can serve as a deceleration lane and storage area for left-turning vehicles at planned crossover points and as a pedestrian rest area.

Flush medians use delineation treatments that do not physically restrict the movement of traffic across the median. The typical type of delineation treatment is painted traffic lanes, but some jurisdictions also use raised pavement markers or mushroom buttons. The principal types of flush medians are narrow divider strips, continuous and alternating left-turn lanes, and two-way left-turn (TWLT) lanes. Flush medians are also described as "painted medians" or "painted left-turn channelization" if left-turn pocket lanes are involved.

The standard design for TWLT lanes is specified by the Manual on Uniform Traffic Control Devices (*2*). The major design requirement of this technique is the median width, which should be at least 3.7 m (12 ft). However it is recognized that 3.0-m (10-ft) widths are common in older urban areas. The intent of a TWLT lane is to remove left-turning vehicles from through lanes and to provide storage in the median area until an acceptable gap in opposing traffic occurs.

The continuous left-turn lane design is similar to the TWLT lane except that it provides individual left-turn lanes for each direction of traffic. This design is also referred to as *side-by-side* *left-turn pocket lanes.* This technique requires a 7.3-m (24-ft)-wide paved median and is not currently in frequent use.

The alternating left-turn lane design provides left-turn opportunity for only one direction of traffic at a time. Both directions of traffic therefore have left-turn capabilities over a limited section of roadway. This design is also described as *back-to-back left-turn pocket lanes.*

Previous research sponsored by NCHRP identified medians and refuge islands as techniques for increasing the safety of pedestrians crossing major arterial streets (*3*). The authors contend, however, that although the potential for increasing safety was present, the actual effect on pedestrian safety was unclear. To emphasize their concern they mention a previous study that claimed to reduce pedestrian accidents by the installation of refuge islands, which on close inspection exhibited problems of regression to the mean (*4*). The literature supports the conclusion of the NCHRP study that there is a substantial lack of definitive information on the effects of medians and refuge islands on pedestrian safety. Those articles that discussed or evaluated medians on roadways is urban and suburban locations were primarily concerned with their impacts on vehicular safety and operations.

The NCHRP study developed a general finding that it is substantially more convenient for pedestrians to cross multilane highways with medians than highways without medians. The authors concluded that medians should be divided as a standard feature of multilane suburban highways (*3*). They cited a study of an arterial street in suburban Virginia that found that almost 90 percent of pedestrian crossings occurred at midblock. It can be expected that when pedestrians are faced with long distances between intersections they will cross at midblock locations to reduce the total walking distance. The presence of medians at these locations can provide a significant benefit to both pedestrian convenience and potential safety on multilane roadways. This is particularly true at those midblock locations with relatively high volumes or unsignalized intersections, since medians greatly simplify the pedestrian's task of crossing the roadway.

The historic use of raised medians and the increased installation rate of TWLT median lanes have resulted in their selection as the predominant median types for the purposes of this paper.

RAISED VERSUS FLUSH MEDIANS

Raised medians were the predominant type of median first used on urban and suburban roadways. Roadway designers considered them effective in controlling left-turn movements, providing a storage space for left-turning vehicles, separating opposing traffic flows, providing an opportunity for aesthetic enhancements, and providing areas for pedestrian refuge. Increased congestion, limited right-of-way, high cost of construction, maintenance costs of raised medians, safety analyses, and the need for increased left-turn opportunities have resulted in the use of flush TWLT median lanes by a large number of agencies. The literature review indicates that TWLT median lanes have been successfully used on urban and suburban roadways with one or more of the following characteristics:

- When traffic volumes are not exceedingly high. There is no firm consensus on the upper-volume threshold at which the advantages of TWLT median lanes dissipate. The ITE survey of practice indicated that the upper level was an average daily traffic (ADT) count of 43,000, whereas other researchers indicated an ADT count of 25,000 (*5,6*).
- On roadways where vehicles make a relatively large number of left turns, commonly in areas with commercial development and frequent driveways. TWLT median lanes have also been successfully implemented in residential areas, combined commercial-residential areas, industrial areas, and in some states, rural areas (*7,8*).
- In areas where the predominant accident patterns are related to left-turn maneuvers and indirect left-turn access cannot be provided with a raised median (*9*).

The advantages and disadvantages of raised and flush medians are summarized in Tables 1 and 2, respectively. These tables were compiled from a 1990 report by Parker (*10*) in conjunction with a 1990 report by Squires and Parsonson (*11*) and are based on a consensus of the literature.

Safety Effectiveness

The majority of the literature reviewed described before-and-after accident studies of TWLT median lanes. Studies that compared the safety effectiveness of raised and flush median types provided mixed results. An inspection of these studies provides an insight into why some of these mixed results occurred.

- Frick (*12*) compared accident rates at two sites during 1968 in Springfield, Illinois. The results of the study indicated that the site with the flush median lane had an accident rate that was 2.65 times greater than that at the site with the raised median section. Since only two sites were used, the study conclusions are questionable because of the small sample sizes.
- Squires and Parsonson (*11*) compared accident occurrences between raised medians and TWLT median lanes in Georgia. They determined that there was no difference in accident rates between the two median types but found that there was a significant difference in the number of accidents per 1.61 km (1 mi). Parker (*13*), in a comparison of 19 raised and 17 flush median sites in Virginia, also determined that there was no significant difference in accident rates between the two median types. That 1983 study determined that the accident rate for raised medians was 275 accidents per hundred million vehicle km (442 accidents per hundred million vehicle mi) and that the rate for flush medians was 380 per hundred million vehicle km (611 per hundred million vehicle mi). Parker also determined that the accident frequency per mile was not significantly different. In a 1990 update to his study Parker (*10*) again determined that neither the accident rates nor the numbers of accidents per mile were significantly different. The studies by Parker (*10*) and Squires and Parsonson (*11*) were, however, too small to experimentally control for differences in traffic volumes, the number of intersections per mile, and the number of driveways per mile between the median types.
- In 1986 Harwood (*9*) analyzed data on accidents at sites in California and Michigan and found that accident rates at TWLT median lanes were 21 to 24 percent lower than accident rates at raised median sections. Harwood used a good experimental approach, but only used sites in California and Michigan with a total raised median length of 35 km (21.8 mi), with a total raised median length of 26 km (16.2 mi) in commercial areas.
- In 1993 Mukherjee et al. (*14*) reported on a comparative analysis of the models developed by Parker (*10*), Squires and Parson-

TABLE 1 Advantages and Disadvantages of Raised Medians *(10,11)*

Advantages:	Disadvantages
1. Discourages new strip development and encourages large planned development.	1. Reduces operational flexibility for emergency vehicles and others.
2. Allows better control of land use by local government.	2. Increases left turn volume at major intersections and median openings.
3. Reduced number of conflicting vehicle maneuvers at driveways.	3. Increases travel time for vehicles desiring to turn left where median openings are not provided.
4. Safer on major arterials with high (>60) number of driveways per mile (>37 driveways per km).	4. Reduces capacity at signalized intersections.
5. Increases traffic flow.	5. Possible increase of accidents at intersections and median openings.
6. Desirable for large pedestrian volumes.	6. Usually increases fixed object accidents.
7. Permits circuitous flow of traffic in grid patterns.	7. Requires motorists to organize their trip making to minimize the need for U-turns and use the arterial only for relatively long through movements.
8. Allows greater speed limits on through road.	8. To minimize delay requires interparcel access, which may not be under government control or would be expensive to purchase and construct.
9. Safer than TWLTL in 4 lane sections.	9. Restricts direct access to adjoining property.
10. Safer than TWLTL in 6 lane sections but depends on number of signals/mile, driveways/mile, ADT, and approaches/mile.	10. Installation costs are higher.
11. Encourages access roads and parallel street development.	11. Can create an over concentration of turns at median openings.
12. Reduces accidents in mid-block areas.	12. Indirect routing may be required for some vehicles.
13. Reduces total driveway maneuvers on the major roadway.	13. When accidently struck, curb may cause driver to lose control of the vehicle.
14. Low maintenance cost of raised medians, depending on final design.	14. A median width of 25 ft (7.6 m) is needed to accommodate U-turns.
15. Studies have shown that delay per left turning vehicle does not increase, up to the studied volume of 3700 vph.	
16. Curbs discourage arbitrary and deliberate crossings of the median.	
17. Reduces number of possible median conflict points.	
18. Provides separation between opposing traffic flows.	
19. Provides a median refuge area for pedestrians.	
20. With raised grass medians, an open space is provided for aesthetics.	

son (*11*), and Harwood (*9*) and revealed conflicting results. The authors concluded that the models and procedures that had been developed were not applicable to all cases and locations.

Although there are problems in many of the studies that compared the safety effectiveness of raised and flush median types, a number of studies determined the safety effectiveness of raised medians and TWLT median lanes without comparison. The raised median safety effectiveness results are summarized below.

- A study of three installations with raised medians by Wooten et al. (*15*) in 1964 determined significant accident reductions; reductions were as high as 69 percent at one site.
- Harwood and Glennon (*16*), using data obtained by Mulinazzi and Michael (*17*), estimated that raised medians would reduce the number of accidents by 50 percent at major intersections and 60 percent of the left-turn accidents at low-volume driveways.
- Harwood (*9*), in his 1986 study, determined that accident rates on roadways with raised medians and four-lane undivided sections were nearly identical after adjustment for the type of development and the number of driveways per mile.

A summary of the safety effectiveness of TWLT median lanes is presented below.

- Sawhill and Neuzil (*18*) in 1963 reported a 25.8 percent decrease in the number of accidents, with only one head-on accident, after a TWLT median lane was installed.
- A 1-year-before and 1-year-after study conducted by Hoffman (*7*) at four sites with TWLT median lanes in Michigan determined that the total number of accidents decreased by 33 percent. The study sites were initially four-lane undivided facilities widened to accommodate the median left-turn lane. Before the installation of the TWLT median lane there were 14 head-on accidents in which 18 people were injured. After the TWLT median lane installation there were eight head-on accidents in which one person was injured.
- A 2-year-before and 2-year-after study was conducted by Thakkar (*19*) on a four-lane roadway on which a TWLT median lane was installed. That study indicated that the total numbers of accidents were reduced by 22.6 percent and that the accident rate was decreased by 27.7 percent.
- Seven sites in Arizona were studied in a 2-year-before and 2-year-after experimental design by Burritt and Coppola (*20*). They

TABLE 2 Advantages and Disadvantages of TWLT Median Lanes (*10,11*)

Advantages:		Disadvantages:	
1.	Left turning vehicles are removed from through traffic while maximum left turning access to side streets and driveways is still provided.	1.	There are conflicting vehicle maneuvers at driveways.
2.	Delay to left turning vehicles and others is often reduced.	2.	Poor operation of roadway if stopping sight distance is less than AASHTO minimum design.
3.	Operational flexibility for emergency vehicles and others is enhanced.	3.	No pedestrian refuge areas for pedestrians free from moving vehicles.
4.	When less than 60 commercial driveways per mile (37 driveways per km) are permitted to be constructed two-way left turn lanes appear to be safer.	4.	Operate poorly under high volume of through traffic.
5.	Roads with two-way left turn lanes are operationally safer than roadways with no separate left turn lanes in the median.	5.	Should not be used when access is required on only one side of the street.
6.	Detours can be easily implemented when required by maintenance in adjacent lanes.	6.	Visibility problem of painted median especially with snow and rain or when pavement markers outlive their design life.
7.	Provides spatial separation between opposing traffic flows.	7.	A safety problem when they are used as a passing lane.
8.	Eliminates the median island fixed object.	8.	High maintenance cost of keeping the pavement striped and raised pavement markers in proper operating condition.
9.	Provides temporary refuge for disabled vehicles.	9.	Must continually instruct the public on proper use and operation.
10.	Can be used as a reversible lane during peak hours.	10.	Delays to left turning vehicles increase dramatically when two way through volume reaches 2800 vpd.
11.	Permits direct access to adjoining properties.	11.	Limits operating speed to a maximum rate 45 mi/h (73 km/h).
		12.	Does not guarantee unidirectional use at high-volume intersections.
		13.	Are not aesthetically pleasing for some people.
		14.	Allows numerous potential traffic conflict points.

determined that the total numbers of accidents were reduced by 35.9 percent and the numbers of head-on accidents were reduced by 66.7 percent after flush median lanes were installed.

- Babcock and Foyle (*21*) examined more than 1,000 accident reports for TWLT median lanes in North Carolina and did not identify any head-on accidents attributed to the median lane.
- In a Virginia study Parker (*13*) determined that 1.05 percent of the accidents on facilities with raised medians were head-on collisions, occurring primarily at the median openings. Parker also determined that 0.98 percent of the accidents on TWLT median sections were head-on collisions with no fatalities involved.

A summary of the safety effectiveness of raised and TWLT median lanes on pedestrians is presented next.

- Billion and Parsons (*22*) reported in a 1962 publication that pedestrians crossing roadways with raised medians had a higher accident rate than those crossing roadways with flush medians. It was not possible to determine from the study, however, if the higher rate for raised medians was due to increased pedestrian activity.
- A 1977 study conducted in London, England, determined that pedestrian refuge islands increased the number of pedestrian accidents (*23*). Problems with the experimental design and the failure to consider changes in traffic and pedestrian volumes result in the questionable validity of the study's conclusions.
- Grayson performed a paired comparison between studies performed in 1962 and 1983 at 75 crossings in London, England (*24*). This comparison determined a reduction in the pedestrian accident rate between the 1962 and the 1983 studies. Because of geometric and traffic control changes that took place between the study periods, it is not possible to ascertain whether the decrease in the number of pedestrian accidents was due to an increase in the number of refuge islands.
- In a 1983 study performed in Virginia, Parker (*13*) determined that 17 of the 1,809 accidents (0.94 percent) occurring during a 3-year period involved pedestrians at raised-median roadway sections. For the TWLT median-lane roadway sections there were 29 pedestrian accidents.

Operational Effectiveness

The majority of the studies reviewed concentrated on the safety effects of medians on vehicular traffic. When operational studies were conducted the measures of effectiveness were speed, travel time, and delay measures. These measures of effectiveness are site specific and are heavily influenced by the number of lanes, type of development, number of driveways, number of intersections, and so on. The following summary groups those studies that had similar results.

- Delay to through vehicles has been determined to be considerably reduced by the use of both raised and flush medians (*6,8,13,25*). Both of these median types remove left-turning vehicles from the through lanes and separate opposing traffic flows.

- Left-turn operations on roadways with raised and flush medians have been determined to have different impacts on operations. Raised medians concentrate left-turn operations at median openings, requiring the driver to select an alternate route or make a U-turn to reach the destination. Harwood (*9*) used a simulation model developed by McCoy et al. (*26*) to compare the operational effectiveness of roadways with raised and flush medians. Harwood determined that the use of raised medians resulted in greater travel time and delay than the use of flush medians.
- Traffic volumes were considered by some researchers as being a warrant for median installation. Stover et al. (*27*) recommended that raised medians be used on all arterial roadways with two or more lanes and traffic volumes of at least 20,000 vehicles per day (vpd). Some researchers (*28,29*) suggested that TWLT median lanes be used when the volume ranged from 10,000 to 25,000 vehicles per day. Volume warrants were opposed by Nemeth (*8*) and others (*6,25*) because successful applications of flush medians were found for volume ranges of between 5,000 and 50,000 vehicles per day. This volume range is typical of the full range of volumes on facilities with four through lanes.
- Research conducted by Parker (*13*) in Virginia and that conducted in other states (*8,21,25*) indicates that TWLT median lanes have been successfully used for posted roadways with speed limits of between 40 km/hr (25 mph) and 89 km/hr (55 mph). TWLT median lanes have been successfully used on some median sections with speeds posted at 97 km/hr (60 mph) (*6,8*).
- Raised medians have resulted in observed wrong way movements when used in highly developed areas (*13,30*).
- Driver confusion and operational efficiency were observed at the openings of raised medians when more than one vehicle occupied the opening at the same time (*13,21*). These occurrences typically happened at unsignalized intersections in heavily developed areas.
- Improperly designed raised-median openings result in U-turn problems (*13,21,27*). The improper design can result in the operators of large vehicles starting their U-turn from the inside through lane instead of the left-turn lane. Some drivers, to avoid running over the curb, must perform a backing maneuver to complete their U-turn.

Installation Criteria

Raised and Flush Medians

- A median of some sort should be used to provide left-turn channelization at all at-grade intersections on high-speed, high-volume roadways (*31*).
- Bretherton et al. (*32*) reported that a raised median is always safer than a TWLT median lane on any four- or six-lane road, regardless of traffic volumes, the number of signals per mile or driveway frequency, or cross-street frequency.
- Squires and Parsonson (*11*) agreed that a raised median is safer than a TWLT median lane on four-lane sections, but claimed that on six-lane roadways with a driveway density of greater than 47/km (75/mi), two or fewer signals per 1.61 km (1 mi), and a maximum of five or six approaches per 1.61 km (1 mi), a TWLT median lane is preferable.
- A raised median works best when there is adequate provision for access between neighboring businesses, such as interconnecting parking lots (*32*).
- Reish and Lalani (*5*) recommended the installation of a raised median when traffic volumes exceed 25,000 vehicles per day.
- The use of some sort of median was recommended by Stover et al. (*27*) on all primary arterials and on secondary arterial roadways with two or more lanes in each direction, average speeds of greater than 56 km/hr (35 mph), and traffic volumes of at least 20,000 vehicles per day. If an existing arterial with a TWLT median lane has a volume of 24,000 to 28,000 vehicles per day, the reconstruction of the arterial to utilize a raised median should be considered, according to Bretherton et al. (*32*).
- Harwood (*9*) found that a four-lane divided facility was more appropriate than an undivided facility for major arterials when the peak flow rate is greater than 1,000 vehicles per hour in one direction and when the driveway density is less than 28/km (45/mi).
- Most agencies prefer to utilize raised or grass-covered flush medians on six-lane arterials (*5*).
- When major driveways or intersections are spaced more than 1.61 km (1 mi) apart, Harwood and Glennon (*16*) suggested that a median barrier be used.
- Parker (*10*) presented a method for selecting between a raised or a painted median.
- Parker (*10,13*) claimed that there is no evidence to limit the use of painted medians to a roadway with a particular volume range or to roadways with a speed limit of under 73 km/hr (45 mph).
- Cribbins et al. (*33*) attempted to use multiple regression to derive an equation for the optimum spacing of median openings but were unable to do so.
- An FHWA implementation package (*4*) reported that traffic-serving businesses appear to be affected by their accessibility to a median crossing. Minimum spacings between median openings were also given.
- Minimum spacings between median openings were also presented by Bretherton et al. (*32*).
- In urban areas, Bretherton et al. (*32*) concluded that median openings could be constructed when the minimum left-turn volume is 500 vehicles per day or 100 vehicles per hour during the peak hour on streets when the speed limit is less than 64 km/hr (40 mph). When the speed limit is over 64 km/hr (40 mph), median openings can be constructed when the minimum left-turn volume is 350 vehicles per day or 70 vehicles per hour during the peak hour.

Two-Way Left-Turn Median Lanes

- The addition of a TWLT median lane to an existing two-way four-lane street reduced the numbers of stops and delays for every combination of volume, average running speed, and left-turn percentage when estimated with a computer model developed by Ballard and McCoy (*34*). Stop and delay reduction isograms are presented. When these are used within the context of a cost-effectiveness analysis they can help to identify when an installation is justified.
- Ballard and McCoy (*35*) also tested 54 combinations of traffic volume, left-turn percentage, and driveway density. In every case the number of stops and the amount of delay were reduced. Those reductions in stops and delay were then used to develop equations to compute the operational benefits of adding a TWLT median

lane. One set of equations was for volumes of less than 800 vehicles per hour; the other set was for volumes greater than this.

• In a similar study conducted by McCoy et al. (*26*) the addition of a TWLT median lane to a two-way two-lane roadway decreased the number of stops and delays for all combinations of volumes and driveway density, with one exception. In this one exception there was no change. Under balanced flow conditions the addition of a TWLT median lane was particularly effective for roadways with volumes of greater than 700 vehicles per hour in each direction and with more than 70 left turns per 305 m (1,000 feet) from each direction. Isograms that could be used within the context of a cost-effectiveness analysis were presented to determine when an installation is justified.

• ITE Committee 5B-4 (*6*) concluded that TWLT lanes are best suited for use on roadways with 40- to 89-km/hr (25- to 55-mph) speed limits in areas of strip development.

• Harwood (*9*) reported, for a roadway with four through lanes, that TWLT median lanes are most appropriate for suburban highways with commercial development, a driveway density of more than 28/km (45/mi), low to moderate volumes of through traffic, high left-turn volumes, and/or a high rate of rear-end or angle accidents associated with left-turn movements.

• The use of a TWLT lane is warranted on arterial highways with an ADT volume of more than 10,000 vehicles per day, average traffic speeds above 48 km/hr (30 mph), a driveway density of more than 37/km (60/mi), fewer than 6 high-volume driveways per km (10/mi), and a left-turn percentage of at least 20 percent of through volume during peak periods, according to Harwood (*16*).

• Bretherton et al. (*32*) reported that TWLT median lanes are definitely warranted on roadways with volumes of more than 28,000 vehicles per day because of the inability of turning vehicles to find acceptable gaps.

• On roadways with four through lanes TWLT median lanes are cost-effective on the basis of operational savings alone, at an ADT volume of 16,200 vehicles per day, according to McCoy et al. (*36*). If accident cost savings are also considered, an installation is justified at volumes of more than 7,100 vehicles per day.

• Thakkar (*19*) also found that TWLT median lanes are safe and cost-effective on roadways with four through lanes as well as on roadways with two through lanes.

• Nemeth (*8*) stated that the use of TWLT median lanes is suitable for roadways with closely spaced driveways and high left-turn volumes, but not when the block lengths are short.

• Stover et al. (*27*) also concluded that TWLT median lanes were suitable for use on roadways with closely spaced driveways, but asserted that they could be effective only if the turning volumes into individual driveways from roadways with a speed limit of 73 km/hr (45 mph) or less were relatively low.

• Walton et al. (*25*) made claims similar to those of Nemeth (*8*), but thought that TWLT median lanes could operate efficiently only under moderate left-turn demands.

• A literature review conducted by Walton and Machemehl (*29*) revealed that a TWLT median lane is preferable to a one-way left-turn lane on roads with four through lanes and ADT volumes of between 10,000 and 20,000 vehicles per day and on roads with two through lanes and ADT volumes of from 4,000 to 12,000 vehicles per day. They also presented tables and equations to be used as guidelines for left-turn lane improvements or installations (*29*).

• The use of TWLT median lanes is not appropriate when there are high pedestrian volumes, the roadway is a major arterial, the block lengths are short, or there are unusual driveway configurations, according to McCoy et al. (*37*).

Refuge Islands

• Dunn (*38*) concluded that refuge islands should be provided if the roadway width exceeds 10 m (33 ft), on the basis of evidence that pedestrians reject headways of less than 4 sec using an average walking speed of 1.2 m/sec (4 ft/sec).

• A 1980 FHWA implementation package (*39*) recommended the consideration of refuge islands on roadways wider than 22.9 m (75 ft).

• A later FHWA implementation package (*40*), published in 1987, stated that a refuge island should be considered when the entire roadway width cannot be crossed within the signal phase at a 1.1-m/s (3.5-ft/s) walking speed and the signal timing cannot be lengthened or an alternate crossing cannot be designated.

• Smith et al. (*3*) recommended the use of refuge islands at locations when medians cannot be provided, traffic speeds are less than 73 km/hr (45 mph), and pedestrian volumes are greater than 100 persons per day. They should not be used for midblock pedestrian crossings across a high-volume streets when speeds are above 73 km/hr (45 mph). Refuge islands should be located every 92 to 153 m (300 to 500 ft).

• Zegeer and Zegeer (*41*) stated that refuge islands are necessary on wide, two-way streets with high vehicular volumes, high speeds, and high pedestrian crossing volumes. They should not be used on narrow streets, when there is a high turning volume of large trucks, when roadway alignment obscures the island, or in areas where snowplowing would be hampered.

STATE-OF-THE-PRACTICE SURVEY

A state-of-the-practice survey was mailed to 150 state and local highway agencies, of which 57 were returned, representing a 38 percent response. The method of analysis followed was to group state and county agencies together and to classify cities by population. The population categories used were 0 to 100,000, 100,000 to 150,000, 150,000 to 500,000, and more than 500,000. The upper and lower boundaries of each class were chosen to give approximately equal numbers of responses in each category.

Regarding the type of warrants or guidelines the agencies used to determine whether or not medians or refuge islands should be installed, the following responses were received. Twenty percent of the states use their own design criteria and 5 percent use the AASHTO Green Book criteria. Factors that states consider include accident history (20 percent), traffic volumes (15 percent), cost (10 percent), number and location of driveways (10 percent), and type of access control (10 percent). Ten percent of the state agencies do not regularly use any guidelines, and 30 percent did not respond to the question.

Mukherjee et al. (*14*) reported in a survey of state highway engineers that 25 percent of the states used some kind of guidelines for median-type decisions, but the material provided was not directly helpful in choosing between a nontraversable median and a TWLT median.

Criteria used by cities with populations of less than 100,000 include accident history (18 percent), AASHTO and Manual on Uniform Traffic Control Devices criteria (*2*) (36 percent), state design criteria (9 percent), and availability of right-of-way (18 percent). Thirty-six percent did not respond to this question.

Cities with populations of between 100,000 and 150,000 consider the following criteria: classification of street (20 percent), available safe gaps (10 percent), AASHTO criteria (10 percent), and the city's own standard plans (10 percent). Thirty percent use no guidelines, and 10 percent did not respond. Cities with populations ranging from 150,000 to 500,000 generally use medians to provide an orderly flow of traffic (20 percent) or install medians with newly constructed arterials (20 percent). Twenty percent do not use any guidelines; 30 percent did not respond.

Large cities (more than 500,000 population) consider traffic volumes (14 percent), pedestrian volumes (14 percent), available right-of-way (29 percent), and arterial classification of street (72 percent) as their criteria. Fourteen percent use their own guidelines.

Pedestrian refuge islands do not receive much attention from roadway agencies. Some agencies do not intend to use medians as pedestrian refuge areas. One agency stated that it does not specifically design medians to be used by pedestrians, although pedestrians use them. Other agencies have low pedestrian volumes, do not account for pedestrians in roadway design, and do not time traffic signal phases to allow pedestrians to cross the entire roadway. In some agencies, however, the needs of elderly and handicapped individuals are currently, or will soon be, included in their specifications for median design.

There was mixed response on the questions concerning acceptable widths for pedestrian refuge islands. Fifty-five percent of the states believe that 1.2 m (4 ft) is an acceptable minimum width for a pedestrian refuge. The results for cities, however, do not concur. The majority of cities in both the 100,000-to-150,000 population range and the 500,000-and-over range believe that 1.2 m (4 ft) is an acceptable minimum width. However only 36 percent of the cities with populations of less than 100,000 believe that 1.2 m (4 ft) is acceptable. Seventy percent of the cities in the 150,000-to-500,000 population range believe that 1.2 m (4 ft) is an unacceptable width for a pedestrian refuge island. All agencies in general believe that pedestrian refuge island widths of 1.8 to 4.9 m (6 to 16 ft) are desirable.

Many different criteria are used to prioritize median and refuge island installations. States typically use accident history (35 percent), traffic volumes (30 percent), or a case-by-case basis (15 percent). Twenty-five percent of the states do not prioritize median installation. City agencies, especially the smaller cities, typically do not prioritize median or refuge island installation. Those that do generally use political considerations, street classification, and traffic and pedestrian volumes. Most agencies do not have any difficulty in using their prioritization procedures. A few agencies commented that installation of a raised median can be a problem if it eliminates left-turn access.

In deciding what factors should be considered in developing new warrants or guidelines for the installation of medians and refuge islands, state officials believe that traffic volumes (65 percent), pedestrian volumes (55 percent), speed (30 percent), accident control (20 percent), number of lanes (10 percent), adjacent land use (10 percent), and the functional classification of the street (10 percent) should be considered. The responses from cities were similar to those from states. Officials in cities with populations of less than 100,000 consider traffic volumes (63 percent), pedestrian volumes (36 percent), street width (36 percent), available gaps (27 percent), and accident history (27 percent). Twenty-seven percent of the cities surveyed did not respond. Forty percent of cities in the 100,000-to-150,000 population range believe that traffic and pedestrian volumes should be considered; 30 percent did not respond. Cities in the 150,000-to-500,000 population range believe that pedestrian crossing time (20 percent) and roadway geometrics (20 percent) should be included, in addition to accident history (20 percent) and traffic volumes (20 percent). Traffic volumes were suggested by 57 percent of the large cities, and pedestrian volumes were recommended by 43 percent.

Most states have their own design specifications for medians. Cities generally use state or AASHTO and ITE guidelines, although some of the larger cities have their own specifications. Some state and city agencies sent copies of their specifications for median construction.

States were almost evenly split on the question of installing different types of medians on the basis of pedestrian use: 45 percent install different types of medians on the basis of pedestrian use, whereas 55 percent, do not. Most cities (at least 60 percent in each population category) do not install different types of medians on the basis of pedestrian use.

Only 10 percent of the states use warrants to determine what type of median should be installed. Nine percent of cities with populations of less than 100,000 use such warrants. None of the other cities use warrants.

Funding for median improvements usually comes from capital improvement funds; special tax districts; federal, state, and local funds; or private development funds. This is true for all states and cities.

Most agencies have not conducted operational studies on medians and refuge islands except for very informal before-and-after studies. A study by the Florida Department of Transportation, however, found that safety for both vehicles and pedestrians was greatly improved when four-lane undivided roads were converted to five-lane roads lanes plus TWLT (four lanes plus a TWLT lane).

In almost all classes of jurisdictions a majority of the agencies believe that flat medians increase pedestrian and vehicular safety. In the class of cities with fewer than 100,000 people, however, 45 percent believe that flat medians do not increase safety and 36 percent believe that flat medians do increase safety. Many agencies commented that flat medians increase vehicular safety, but not pedestrian safety, since they offer no physical protection from vehicular traffic (unlike raised medians).

CONCLUSIONS

Although the results of the safety analyses on medians and refuge islands are mixed, it appears that both raised and TWLT medians significantly reduce the number and severity of vehicular accidents. The literature review made it apparent that both raised and TWLT medians offer significant vehicular accident reductions and over those for comparable roadways without medians and offer vehicular benefits. Typical reductions in the total number of vehicular accidents for both median types are in the 25 to 35 percent range.

The literature did not provide a conclusive indication that medians improved pedestrian safety. This was due to the small number of pedestrian accidents encountered during the studies.

Both raised and TWLT medians result in a reduction in accident severity. The results were mixed with regard to whether raised or TWLT medians decreased accident severity by the same amount. Some researchers concluded that raised medians reduced vehicular accident severity slightly more than TWLT median lanes. Another researcher found that there was no discernible difference in the accident severity between roadways with raised medians and those with TWLT median lanes.

Rear-end and head-on accidents decreased with both raised median and TWLT median lane installation. More fixed-object and U-turn accidents occur on roadways with raised medians than on those with TWLT median lanes. Significantly higher numbers of midblock left-turn accidents occur on roadways with TWLT median lanes than on those with raised medians. The initial concern of researchers that TWLT median lanes would result in a larger number of head-on accidents was not determined to be true. Roadways with raised medians and TWLT median lanes have similar head-on accident experiences. The head-on accidents on roadways with raised medians occur at the median crossover points.

The current literature suggests that both raised and TWLT median lanes can be used on roadways with posted speed ranges of 40 to 89 km/hr (25 to 55 mph) and all volume ranges typically encountered on urban and suburban arterials. The use of raised medians results in more delay and travel time because of the need for U-turns to reach destination points. TWLT median lanes are appropriate for suburban roadways with commercial development and driveway densities greater than 28/km (45/mi).

The state-of-the-practice survey revealed that there is no universal set of factors that can be used to determine the need to install medians. Whereas states rely on accident history, traffic volumes, numbers and locations of driveways, type of access control, and cost, the larger cities rely on traffic volumes, pedestrian volumes, available right-of-way, and street classification. A greater divergence was found in the smaller cities.

The research described here revealed that there is a need to develop a definitive set of guidelines that can be used by cities and states to determine the most appropriate median treatment on arterials. These guidelines must be based on safety as well as operational criteria.

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Analysis of Influence of Perception-Reaction Time on Case III Intersection Sight Distance

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For years researchers have attempted to conclusively define values to be used for perception-reaction (PR) times for highway design and operations. Several empirically based studies have confirmed values that are used in different design guides and manuals. However there still appears to be no final agreement on what values should be used for various design purposes. PR time is an essential element in determining intersection sight distance (ISD) requirements. Although there has been much discussion of the appropriate values of PR times that should be used for this purpose, an understanding of the relative influence of PR time on ISD would help to determine whether very exact PR times are needed to adequately design intersections. Previous work on the assessment of appropriate PR times is reviewed, and the influence of varying PR times on the determination of Case III ISD values is analyzed to see whether the designer needs to be concerned about the selection of exact PR times. Several conclusions were reached on the bases of the review and the analyses. It was concluded that although there has been a great deal of research on PR times, there appears to be some doubt about how well the current values used for highway design purposes represent real-world conditions. Also for applications related to Case III ISD determination PR time has little influence on the overall ISD requirement. On the basis of these conclusions it is recommended that the current values used for PR time for Case III ISD applications be retained because of their validation from several empirical studies and the insensitivity in change of ISD values relative to changes in PR times.

Driving a vehicle safely through an intersection is a complex job requiring the coordination of sensory, perceptual, cognitive, and motor skills. Complicating the smooth interaction of these tasks are outside influences, such as intersection geometry, and the presence of external factors, such as other vehicles or pedestrians.

In attempting to design safer intersections, an area of importance in the highway geometric area has been sight distance. The AASHTO publication *A Policy on the Geometric Design of Streets and Highways* (*1*), often referred to as the Green Book, is the principal guidance for highway design in the United States, and it details the processes for determining sight distances for a variety of operational situations. Sight distance is necessary to ensure safe vehicle operations related to stopping, intersection movements, and passing situations.

Intersection sight distance is the unobstructed line of sight sufficient to allow approaching drivers to anticipate and avoid potential conflict situations at intersections. There are four intersection situations of interest as described by AASHTO.

- Case I—no control,
- Case II—yield control on minor road,
- Case III—stop control on minor road, and
- Case IV—signal control.

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In addition, AASHTO has separated Case III into three subcases dealing with different intersection maneuvers.

- Case IIIA—crossing maneuver,
- Case IIIB—turning left into a major highway, and
- Case IIIC—turning right into a major highway.

A factor used in the determination of intersection sight distance (ISD) that has received a great deal of attention is perception-reaction (PR) time. PR time has generally been thought to be the time needed to perceive a stimulus and, if necessary, the additional time required to take some type of action in response. Numerous studies have attempted to analyze what has simply been called perception and reaction for the purpose of determining a value to be used in roadway design. For ISD situations, AASHTO has established the value of PR times to be used in the various equations that define ISD. Case I uses a PR time of 2.0 sec plus an additional 1.0 sec to adjust speed. The ISD necessary for Case II situations is stopping sight distance (SSD) on the minor roadway. Therefore the PR time is set at 2.5 sec. All Case III situations use a value of 2.0 sec.

Over the years there has been a great deal of discussion of what is an appropriate PR time for use in highway design purposes. This paper reviews this previous work and analyzes the influence of varying PR times on the determination of Case III ISD values to see whether the designer needs to be concerned about the selection of a very exact PR time.

BACKGROUND

A great deal of effort has been expended for the purpose of arriving at a single value for PR time that neatly encompasses the entire driving population. Although AASHTO currently recommends an ISD PR time of 2.0 sec, past research has questioned the use of that number.

The first formal discussion of ISD was published in 1940 in *A Policy on Intersections at Grade* (*2*). The formulas presented in that text used a PR time of 2.0 sec. That value appears to be a direct result of the 1940 AASHO publication *A Policy on Sight Distance for Highways* (*3*). However no explanation other than "simplicity" was provided in the 1940 policy for the assumed value of 2.0 sec. Having only this rather arbitrary determination of a PR time for ISD cases, researchers have sought to define an appropriate PR time more scientifically.

One approach for calculating a PR time has been to measure the durations of several of the constituent elements and to sum these time segments to determine a value. Another approach has been to calculate percentile values for the measurable elements and assemble these percentile values into some single value or range of values for PR time.

An example of this general approach can be found in the work of Hooper and McGee (*4*). In trying to validate the 2.0 sec used for PR time in a Case III ISD scenario, the authors conducted an experiment that measured subprocesses that make up the aggregate PR time. For Case III ISD they defined these elements as head and eye movement, fixation, decision, and reaction. Values are presented for each of these subprocesses and then summed (Table 1). Adding these values yields a total PR time for Case III ISD of 2.21 sec. This value is 0.21 sec above the current values, an increase of 10.5 percent. The authors state that the criterion of 2.0 sec should be retained, but that the formulation of the PR time should be redefined, presumably to their model.

This type of approach has been criticized for having two principal drawbacks. The first is that of the implicit assumption that the elements of the process act in series without any time overlap or parallel functioning. The second is that it is highly unlikely that an individual will consistently perform at or near prespecified percentile values for all of the individual elements of the process (*5*).

Additional attempts to determine PR times have focused on empirical studies of the entire PR process. In those studies measurements are made of the time from the onset of the perception process through the completion of the reaction component and the onset of mechanical acceleration or braking. However many of those studies tend to be deficient in that the subjects are already alerted to the fact that their reactions will be tested.

An example of this approach can be found in the work of Hostetter et al. (*6*). As part of a study examining the different Case III scenarios, subjects were observed while trying to complete all three Case III intersection maneuvers. PR times were measured as the time from the first head movement after a stop to the application of the accelerator. On the basis of the results of those experiments a recommendation was made to keep the current specification of 2.0 sec for Case IIIA, but to change the specification for Cases IIIB and IIIC to 2.5 sec.

ANALYSES

Although researchers do not seem to be able to convincingly debunk the current value for PR time, this does not stop discussions of what is an appropriate PR time to use for highway design situations. Although exchanges concerning what particular value of PR time is appropriate are informative, the nature of the AASHTO ISD equations could render these discussions moot. An appreciation of the relative influence of PR time on ISD values would help to determine whether very exact PR values are needed to properly design intersections.

To test this point the impact of PR time on each of the Case III sight distance equations was evaluated in the following series of analyses of each of the Case III maneuvers. Each analysis consists of a brief description of the AASHTO equation and then sensitivity analyses of the various parameters of interest. The sensitivity analyses were carried out in two parts. The first set of sensitivity analyses varied one parameter at a time, holding the others to appropriate default values. The second set of analyses examined the effects of varying two parameters at the same time. The measure of effectiveness used in both cases was elasticity. Elasticity is a concept used in economics to relate one parameter to another. In economic theory elasticity is the slope of the demand-price curve at a given time point. Elasticity is a measure of the change in demand for a unit change in price. The economic elasticity is weighted by the equilibrium point of the demand-price curve. In essence the elasticity is a measure of the sensitivity of the demand curve to price. In the application of this paper the sensitivity of the AASHTO equations are measured with respect to the parameter of PR time. These sensitivities are weighted by the mean. Because both sensitivities measure the change in one variable with respect to a second independent variable, the term *elasticity* is adopted and used throughout this paper as a single-figure measure of an equation's sensitivity (*7*). The elasticity values computed in the sensitivity analyses for this paper are the ratio of change in ISD over the range of interest to the mean ISD divided by the ratio of change in the selected parameter over the range of interest to the mean of that parameter.

TABLE 1 85th Percentile Values for PR Subprocesses for ISD Case III (*4*)

Element	Time (s)
Head and eye movement	0.24
Fixation	0.20
Decision	0.85
Reaction	0.92
Total	2.21

CASE IIIA—CROSSING MANEUVER

The AASHTO Green Book states that the sight distance for a crossing maneuver is based on the time it takes for the stopped vehicle to clear the intersection and the distance that a vehicle will travel along the major road at its design speed in that amount of time. In reality, however, the time element also includes a perception-reaction and vehicle transmission actuation component. The sight distance for Case IIIA is calculated from the following equation:

$$\text{ISD} = 0.2784\,V(J + t_a) \qquad (1)$$

where

ISD = d_1 (to the left of the vehicle on the minor road) or d_2 (to the right of the vehicle on the minor road) sight distance along the major highway from the intersection (m);
V = design speed of the major highway (km/hr);
J = sum of the perception time and the time required to actuate the clutch or actuate an automatic shift (sec);
t_a = time required to accelerate and traverse the distance (S) to clear the major highway pavement (sec);
$S = D + W + L$, the distance that the crossing vehicle must travel to clear the major highway (m);

D = distance from the near edge of pavement to the front of a stopped vehicle (m);
W = pavement width along the path of the crossing vehicle (m); and
L = overall length of the vehicle (m).

ISD is measured from a driver eye height of 1.0675 m (for passenger cars) to the top of an object 1.2963 m (nominally the overall height of another passenger car) above the pavement.

The J term, or PR time, is the time allowed for scanning in both directions by the vehicle operator to determine whether there is a sufficient gap to initiate and complete the crossing maneuver safely and the time to actuate the transmission. According to AASHTO the value for J is equal to 2.0 sec. This value is a constant since there is no guidance as to when it might be appropriate to vary J for changing conditions, such as operator or vehicle types. However the key issue here is not what value of PR time is absolutely correct (e.g., 2.0 or 2.5 sec) but what the impact of different values of PR time on the ISD values would be.

To answer this question sensitivity analyses were performed on the Case IIIA formulation. As mentioned above the measure of effectiveness used in all analyses was elasticity. Again the elasticity values are the ratio of change in ISD over the range of interest to the mean ISD divided by the ratio of change in the selected parameter over the range of interest to the mean of that parameter. From Figure 1, for example, the elasticity value (E_d) for PR time is calculated as

$$E_d = \frac{\dfrac{302.6515 - 235.399}{\dfrac{302.6515 + 235.399}{2}}}{\dfrac{3.5 - 0.50}{\dfrac{3.5 + 0.50}{2}}} = \frac{\dfrac{302.6515 - 235.399}{302.6515 + 235.399}}{\dfrac{3.5 - 0.50}{3.5 + 0.50}} = 0.17 \qquad (2)$$

The result of the sample computation shown above indicates that ISD will change by 0.17 percent for each 1.00 percent change in PR time over the range of interest for PR time. Because the relationship between most parameters and ISD is nonlinear, the sensitivity of ISD to the parameter is not constant over the range of interest. However the elasticity value is the most accepted way of representing the relative magnitude of that sensitivity in a single number. A positive value for E_d indicates that ISD increases with increasing values of the parameter. A negative value of E_d indicates that ISD decreases with increasing values of the parameter.

Sensitivity analyses were performed on the 1990 AASHTO Case IIIA procedure to determine the relative importance of the various factors that are part of the ISD equation. Different ISD values were calculated as a variable of interest was stepped through a range of values and the remaining variables were held constant at some predefined default value. A spreadsheet was used to perform the calculations and tabulate the results. For a single-parameter analysis, for example, a set of ISD values was calculated for a series of design speeds ranging from 32.2 km/hr (20 mph) to 112.7 km/hr (70 mph) in 16.1-km/hr (10-mph) increments while the value of t_a was held constant at 10.0 sec and the value of J was held at 2.0 sec. Figure 1 shows the table and the results of the first set of sensitivity analyses.

When design speed was varied and all other variables were held constant, ISD showed a sensitivity to changes in design speed characteristic of a variable multiplied by a constant. The elasticity of the relation between the two variables is equal to 1.00. This means that for 1.00 percent change in design speed there is a corresponding change of 1.00 percent in ISD.

A second analysis of the current model was performed by varying t_a through a series of values ranging from 4.00 to 16.00 sec in 0.25-sec increments while the value of design speed was held constant at 80.5 km/hr (50 mph) and the value of J was held at 2.0 sec. The elasticity of the relation between ISD and the time t_a is equal to 0.75. This means that for a 1.00 percent change in the time required to accelerate and clear the major highway pavement there is a corresponding change of 0.75 percent in ISD. This means that the ISD model is not quite as sensitive to changes in t_a as it is to changes in design speed.

The current model was analyzed a third time by varying J through a series of values ranging from 0.50 to 3.50 sec in 0.10-sec increments while the value of design speed was held constant at 80.5 km/hr (50 mph) and the value of t_a was held at 10 sec. The elasticity of the relation between ISD and the perception/shift actuation time is equal to 0.17. This means that for a 1.00 percent change in PR time there is a corresponding change of 0.17 percent in ISD (Figure 2). This means that the ISD model is relatively insensitive to changes in J.

Figure 3 shows the revised criteria that would result if PR time were raised from 2.0 to 2.5 or 3.0 sec. Increasing the PR time by 0.5 sec to 2.5 sec would increase the ISD required for Case IIIA by only 4.17 percent. An increase of 1.0 sec in PR time, to 3.0 sec, would increase Case IIIA ISD by 8.3 percent.

Although the single-parameter sensitivity analysis completely tests each individual parameter's influence on ISD, it is possible that varying combinations of the parameters might yield unexpected results. For this reason analysis of a second set of sensitivities, which varied both t_a and J within a given design speed, was undertaken. Speed was varied from 32.2 km/hr (20 mph) to 112.7 km/hr (70 mph) in 16.1-km/hr (10-mph) increments. The J term was varied from 0.5 to 3.5 sec in 0.1-sec increments. The time to accelerate, t_a, was varied from 4.00 to 16.00 sec in 0.25-sec increments. A tabulation (Table 2; see below as well) of the elasticity values was produced and used to construct the surface plot shown in Figure 2. The plot shows the elasticities that result from varying V and t_a across a range of values for J (0.5 to 3.5 sec).

An interesting aspect of this plot is that the elasticities are the same across any design speed. For example the elasticity value obtained with V = 32.2 km/hr (20 mph) and t_a = 8.0 sec while varying J is equal to 0.18. For all other values of V the E_d is also 0.18 when t_a = 8.0 sec. This occurs because the ISD for Case IIIA is directly proportional to speed. Since the denominator (the J terms) of the elasticity equation (Equation 2) remains constant, the relative proportion between the elasticity values is directly proportional to speed. As expressed earlier, because elasticity is a measure of the range over the mean of one parameter, a proportional increase will simply cancel out.

It should be pointed out that over the range of all possible conditions for V, t_a, and J the elasticities vary from 0.33 to 0.11. [Table 2 shows the range of elasticity values for V = 32.2 km/hr (20 mph). Table 2 would be the same for all values of V, as explained above.] This would lead one to believe that PR time can have a greater influence on ISD values than was illustrated. However these endpoint values are for design and operating conditions that are relatively extreme. Therefore the characterization that ISD is relatively insensitive to PR time holds true.

ISD=0.2784*V*(ta+J)

Default V = 80.5 kph
Default ta = 10 sec
Default J = 2 sec

Ed =	1.00	Ed =	0.75	Ed =	0.17
V (kph)	ISD (m)	ta (sec.)	ISD (m)	J (sec.)	ISD (m)
0	0.0	0	44.8	0	224.2
32.2	107.6	4.00	134.5	0.50	235.4
40.3	134.5	4.25	140.1	0.60	237.6
48.3	161:4	4.50	145.7	0.70	239.9
56.4	188.3	4.75	151.3	0.80	242.1
64.4	215.2	5.00	156.9	0.90	244.4
72.5	242.1	5.25	162.5	1.00	246.6
80.5	269.0	5.50	168.1	1.10	248.8
88.6	295.9	5.75	173.7	1.20	251.1
96.6	322.8	6.00	179.3	1.30	253.3
104.7	349.7	6.25	184.9	1.40	255.6
112.7	376.6	6.50	190.5	1.50	257.8
		6.75	196.2	1.60	260.0
		7.00	201.8	1.70	262.3
		7.25	207.4	1.80	264.5
		7.50	213.0	1.90	266.8
		7.75	218.6	2.00	269.0
		8.00	224.2	2.10	271.3
		8.25	229.8	2.20	273.5
		8.50	235.4	2.30	275.7
		8.75	241.0	2.40	278.0
		9.00	246.6	2.50	280.2
		9.25	252.2	2.60	282.5
		9.50	257.8	2.70	284.7
		9.75	263.4	2.80	286.9
		10.00	269.0	2.90	289.2
		10.25	274.6	3.00	291.4
		10.50	280.2	3.10	293.7
		10.75	285.8	3.20	295.9
		11.00	291.4	3.30	298.2
		11.25	297.0	3.40	300.4
		11.50	302.6	3.50	302.6
		11.75	308.2		
		12.00	313.8		
		12.25	319.4		
		12.50	325.1		
		12.75	330.7		
		13.00	336.3		
		13.25	341.9		
		13.50	347.5		
		13.75	353.1		
		14.00	358.7		
		14.25	364.3		
		14.50	369.9		
		14.75	375.5		
		15.00	381.1		
		15.25	386.7		
		15.50	392.3		
		15.75	397.9		
		16.00	403.5		

FIGURE 1 Single-parameter sensitivity analysis of AASHTO Case IIIA.

CASE IIIB—LEFT-TURN MANEUVER

In the AASHTO Case IIIB situation a vehicle is stopped on the minor road. The intention of the driver is to complete a left-turn maneuver by clearing traffic approaching from the left and then entering the traffic stream approaching from the right. The AASHTO ISD model is constructed such that a vehicle accelerating from a stop to turn left into a major highway should have, as a minimum, sufficient sight distance so that a collision will not occur if a vehicle approaching from the right and traveling at the design speed of the major road appears when the turning vehicle begins its maneuver. The turning vehicle should also be able to accelerate to a safe running speed by the time the approaching vehicle closes to within a specified tailgate distance or minimum separation. According to the 1990 Green Book it is assumed that the major-road vehicle reduces speed from the design speed to 85 percent of the design speed of the major road.

The required intersection sight distance in Case IIIB is given by

$$ISD = Q - h \tag{3}$$

where

ISD = sight distance along the major highway from the intersection required for a vehicle to depart from a stop, accelerate to a speed V_a, and complete a turn to the left without being overtaken by a vehicle approaching from the right traveling at design speed and decelerating to a speed V_a(m);

Q = distance traveled by major-road vehicle approaching from the right (m); and

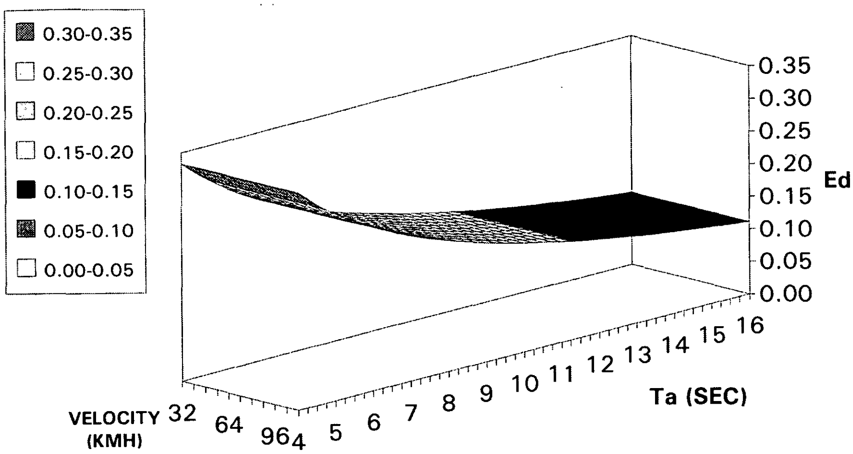

FIGURE 2 Surface plot of multiple-parameter sensitivity analysis for Case IIIA.

h = distance on major roadway traveled by minor-road vehicle from the midpoint of the turning lane on the minor roadway to end of maneuver (m).

In both the 1990 Green Book and previous editions the underlying assumptions used in developing the equations for Q and h were not clearly explained or defined. Information received during the course of research in NCHRP 15(14)-1 identified several of these assumptions and allowed the graphical solutions in the Green Book to be replicated exactly. For the Case IIIB maneuver Q and h can be expressed as

$$Q = 0.2784(J + t_a) \times 0.95V_{ds} \tag{4}$$

where V_{ds} is design speed of the major roadway (km/hr), and

$$h = P - 4.88 - \mathrm{VG} - L \tag{5}$$

where P is distance required for minor road vehicle to reach 85 percent of design speed (m) and VG is vehicle gap at conclusion of maneuver (m). AASHTO specifies that VG is equal to the distance traveled in 2.0 sec at 85 percent of the design speed of the major roadway.

$$VG = 0.2784\ (0.85V_{ds}t_{VG}) \tag{6}$$

where t_{VG} is specified vehicle gap (2.0 sec).

In this current formulation two assumptions are made regarding the major-road vehicle. The first is that the driver of the major-road vehicle is traveling at the design speed of the roadway. This is a reasonable assumption because in some situations motorists may drive above the posted speed limit of the facility, operating nearer the design speed. In those cases in which drivers are traveling at less than the design speed, this assumption creates a margin of safety by prescribing more sight distance than is actually required.

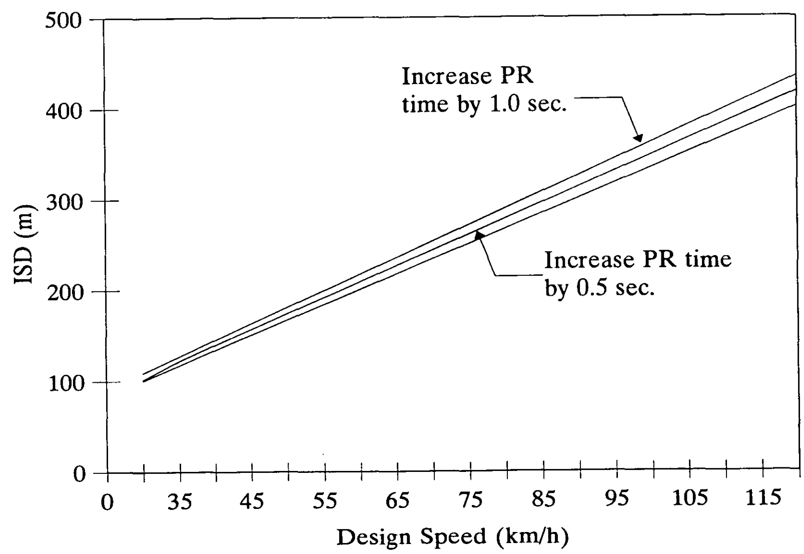

FIGURE 3 Effect of change in PR time on ISD for Case IIIA.

TABLE 2 Elasticity Values for Case IIIA Sensitivity Analysis

Velocity (km/h)	t_a (s)	Elasticity	Velocity (km/h)	t_a (s)	Elasticity
32.2*	4.00	0.3333	32.2	10.25	0.1633
32.2	4.25	0.3200	32.2	10.50	0.1600
32.2	4.50	0.3077	32.2	10.75	0.1569
32.2	4.75	0.2963	32.2	11.00	0.1538
32.2	5.00	0.2857	32.2	11.25	0.1509
32.2	5.25	0.2759	32.2	11.50	0.1481
32.2	5.50	0.2667	32.2	11.75	0.1455
32.2	5.75	0.2581	32.2	12.00	0.1429
32.2	6.00	0.2500	32.2	12.25	0.1404
32.2	6.25	0.2424	32.2	12.50	0.1379
32.2	6.50	0.2353	32.2	12.75	0.1356
32.2	6.75	0.2286	32.2	13.00	0.1333
32.2	7.00	0.2222	32.2	13.25	0.1311
32.2	7.25	0.2162	32.2	13.50	0.1290
32.2	7.50	0.2105	32.2	13.75	0.1270
32.2	7.75	0.2051	32.2	14.00	0.1250
32.2	8.00	0.2000	32.2	14.25	0.1231
32.2	8.25	0.1951	32.2	14.50	0.1212
32.2	8.50	0.1905	32.2	14.75	0.1194
32.2	8.75	0.1860	32.2	15.00	0.1176
32.2	9.00	0.1818	32.2	15.25	0.1159
32.2	9.25	0.1778	32.2	15.50	0.1143
32.2	9.50	0.1739	32.2	15.75	0.1127
32.2	9.75	0.1702	32.2	16.00	0.1111
32.2	10.00	0.1667			

*NOTE: 32.2 km/h = 20 mi/h.

The second assumption concerning the major-road vehicle is that the driver decelerates to 85 percent of the design speed on the major roadway. This may not be a valid assumption for all drivers at all locations because it is not known what the average or expected speed reduction would be across the general population. The use of the 85 percent reduction in design speed results in the use of a multiplier of 0.95 in Equation 4. It is assumed that the driver of the major-road vehicle initiates braking at 100 percent of design speed and concludes the braking at 85 percent of design speed. The factor of 0.95 indicates a slowing of the vehicle over the braking distance. The AASHTO Green Book does not indicate how the 0.95 was developed. It is assumed to be a close approximation of an average speed over the braking distance.

Sensitivity analyses were performed on the 1990 AASHTO Case IIIB procedure. As in the other AASHTO procedures the purpose of the analyses was to determine the relative importance of the various factors that are present in the Case IIIB equation. Different ISD values were calculated as the variable of interest was stepped through a range of values. While one variable was being varied, the other variables were held constant at a predefined default value.

For the AASHTO Case IIIB equation the predefined defaults were set at V = 80.5 km/hr (50 mph), J = 2.00 sec, L = 15.795 m (19 ft), and t_{VG} = 2.00 sec. The V term was stepped in 8.05-km/hr (5-mph) increments from 32.2 km/hr (20 mph) to 112.7 km/hr (70 mph), J was stepped in 0.25-sec increments from 0.50 to 3.50 sec, and t_{VG} was stepped in 0.10-sec increments from 0.30 to 3.00 sec. Because the Case IIIB equation requires the additional parameters of P and t_a, these values were created in a lookup table in the spreadsheet. This table is based on Table IX-7 of the 1990 Green Book (*1*). Because Table IX-7 did not contain all the needed values, several intermediate points were found by interpolation. The lookup table contains values for P and t_a for a range of from 24.15 km/hr (15 mph) to 112.7 km/hr (70 mph). The lookup table is entered with the current value of V, the velocity, and the corresponding values for the parameters P and t_a are found.

Figure 4 shows the results of the sensitivity analysis for the AASHTO formulation of Case IIIB. When design speed was varied and all other variables were held constant, ISD showed a great sensitivity to changes in design speed. The elasticity of the relation between the two variables is equal to 1.36. This means that

$$ISD = Q - h$$
$$Q = 0.2784*(0.95V)*(ta + J)$$
$$h = P - 4.88 - VG - L$$
$$VG = 0.2784\ (0.85)*V*tvg$$

Default V = 80.5 kph
(P = 155.55 m, ta = 14.9 sec)
Default J = 2 sec
Default L = 5.795 m
Default tvg = 2 sec

Ed =	1.36	Ed =	0.17	Ed =	0.13
V (kph)	ISD (m)	J (sec.)	ISD (m)	tvg (sec.)	ISD (m)
0		0	210.6	0	215.0
32.2	66.3	0.50	221.2	0.30	220.8
40.3	87.6	0.60	223.3	0.40	222.7
48.3	110.2	0.70	225.5	0.50	224.6
56.4	141.7	0.80	227.6	0.60	226.5
64.4	173.4	0.90	229.7	0.70	228.4
72.5	210.7	1.00	231.9	0.80	230.3
80.5	253.1	1.10	234.0	0.90	232.2
88.6	300.7	1.20	236.1	1.00	234.1
96.6	351.8	1.30	238.2	1.10	236.0
104.7	411.4	1.40	240.4	1.20	237.9
112.7	479.9	1.50	242.5	1.30	239.8
		1.60	244.6	1.40	241.7
		1.70	246.8	1.50	243.6
		1.80	248.9	1.60	245.5
		1.90	251.0	1.70	247.4
		2.00	253.1	1.80	249.3
		2.10	255.3	1.90	251.2
		2.20	257.4	2.00	253.1
		2.30	259.5	2.10	255.1
		2.40	261.7	2.20	257.0
		2.50	263.8	2.30	258.9
		2.60	265.9	2.40	260.8
		2.70	268.1	2.50	262.7
		2.80	270.2	2.60	264.6
		2.90	272.3	2.70	266.5
		3.00	274.4	2.80	268.4
		3.10	276.6	2.90	270.3
		3.20	278.7	3.00	272.2
		3.30	280.8		
		3.40	283.0		
		3.50	285.1		

FIGURE 4 Single-parameter sensitivity analysis of AASHTO Case IIIB.

for every 1.00 percent change in design speed there is a corresponding change of 1.36 percent in the ISD value.

When the equation is examined with respect to J, the PR time, the elasticity value is 0.17. This means that the ISD is relatively insensitive to changes in the PR time. A 50.0 percent change in J, an increase from 2 to 3 sec, results in an increase in the ISD values of only 8.5 percent. Similarly when the equation is varied with respect to t_{VG}, the elasticity is 0.13, again indicating a relatively inelastic parameter.

As with Case IIIA a second set of sensitivity analyses was performed to rule out the possibility that varying combinations of the input parameters would yield unexpected results. These analyses varied V, J, and t_{VG}. The V term was varied from 40.25 km/hr (25 mph) to 112.7 km/hr (70 mph) in 8.05-km/hr (5-mph) increments. The J term was varied from 0.5 to 3.5 sec in 0.1-sec increments. The t_{VG} term was varied from 0.3 to 3.0 sec in 0.1-sec increments. A surface plot of the results is shown in Figure 5. It should be noted that elasticities are not plotted for V equal to 32.2 km/hr (20 mph) because of the tendency of the AASHTO equation to yield negative numbers at parameter combinations of low speed, quick reaction time, and short tailgate distances.

The highest elasticity plotted in Figure 5 is approximately 0.56. This particular point results from the evaluation of the AASHTO equation with V = 40.25 km/hr (25 mph), t_{VG} = 0.3 sec, and J being varied from 0.5 to 3.5 sec. The lowest elasticity plotted in Figure 5 is 0.14, indicating that for every 1.00 percent increase in PR time there is a 0.14 percent increase in ISD. This means that if the PR time for Case IIIB was increased from 2 to 4 sec, a 100 percent increase, the resulting ISD would increase only 14 percent. This case resulted from an evaluation of the AASHTO equation with V = 112.7 km/hr (70 mph), t_{VG} = 3.0 sec, and J being varied from 0.5 to 3.5 sec. It should be noted that the higher elasticity value is for a set of design and operating conditions that is relatively excessive. As with Case IIIA the characterization that ISD is relatively insensitive to PR time holds true.

Following on the previous discussion, an examintion of Figure 5 shows that the majority of the elasticities are in the range of 0.1 to 0.2. The shading for this range of elasticities covers approximately 50 percent of the surface. If elasticities in the range of 0.2 to 0.3 are included, approximately 75 percent of the surface is covered. This again illustrates, for most design and operational conditions, that the Case IIIB AASHTO equation is relatively insensitive to PR time. In particular the typical range of Case IIIB situations, having a design speed of 48.3 km/hr (30 mph) to 80.5 km/hr (50 mph) and a t_{VG} set by AASHTO equal to 2.0 sec, lies well within the first shaded region, with elasticities in the range of 0.1 to 0.2.

Figure 6 shows the ISD values that would result if the PR time for Case IIIB were increased by either 0.5 or 1.0 sec. With an

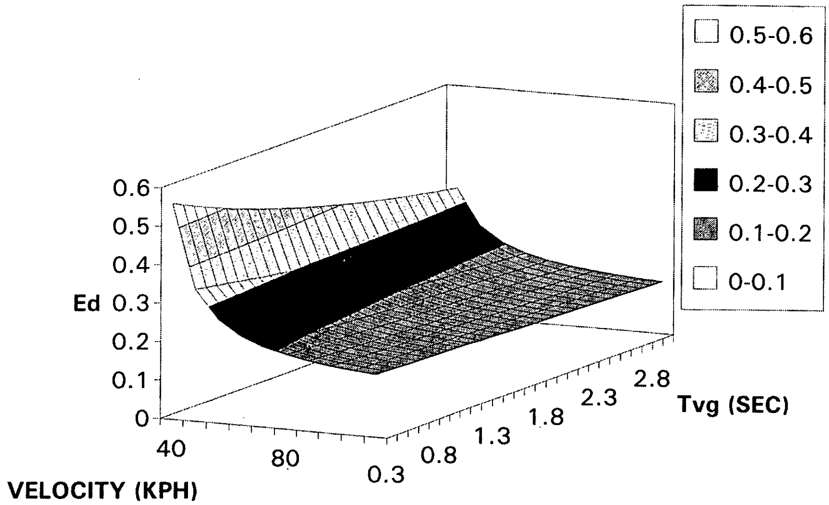

FIGURE 5 Surface plot of multiple-parameter sensitivity analysis for Case IIIB.

increase to 2.5 sec the change in ISD ranges from 6 percent at 32.2 km/hr (20 mph) to 3 percent at 112.7 km/hr (70 mph). If PR time was increased to 3.0 sec the ISD would increase by 11 percent at 32.2 km/hr (20 mph) and 6 percent at 112.7 km/hr (70 mph).

CASE IIIC—RIGHT-TURN MANEUVER

According to current AASHTO policy the Case IIIC scenario is the same as the Case IIIB scenario, only the vehicle on the minor road is making a right turn instead of a left turn. As such the minor-road vehicle has to be concerned only with major-road vehicles approaching from one direction. However the equations for calculating the sight distance are nearly identical, resulting in Case IIIC values that differ by approximately two feet from the corresponding values in Case IIIB. The only difference in the calculations is that the constant value of 4.88 m (16 ft) in Equation 5 is replaced by a value of 4.3615 m (14.3 ft), reflecting the shorter distance traveled by the minor-road vehicle when making a right turn.

The sensitivity analyses of Case IIIC yield the same results as those for Case IIIB. Varying a single parameter at a time results in an elasticity of 0.17. Varying two parameters at a time, to check for any unusual occurrences, produces values that, when plotted, create a duplicate of Figure 5 (from the Case IIIB discussion).

CONCLUSIONS AND RECOMMENDATIONS

Several conclusions can be reached on the basis of the results of the review and analyses. These include the following:

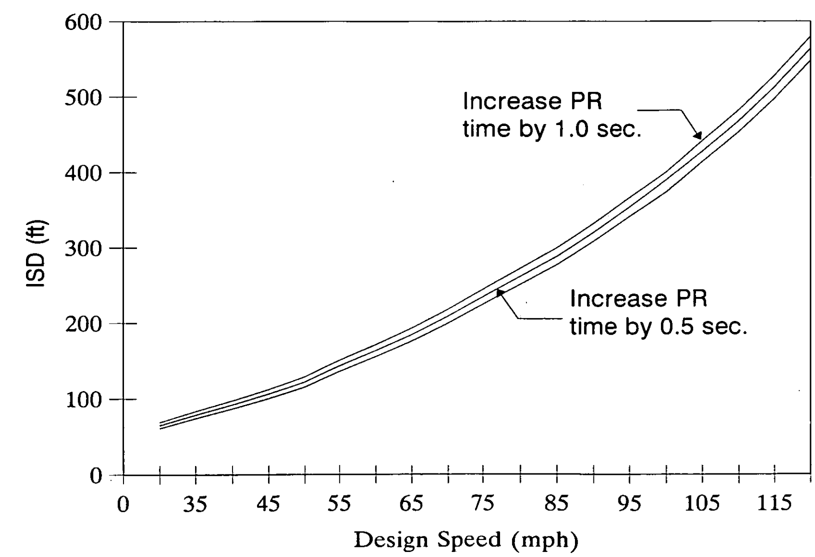

FIGURE 6 Effect of change in PR time on ISD for Case IIIB.

- PR time has long been considered an important element in highway design and operations.
- Several empirically based studies have confirmed the use of the currently specified AASHTO values for PR times for ISD applications.
- Although there has been a great deal of research on PR times, especially as this topic relates to highway design, there appears to be some doubt about how well the current values used for highway design purposes represent real-world conditions.
- For applications related to Case III ISD determination, PR time has little influence on the overall ISD requirement.

Following on these conclusions, the following recommendations are made:

- The current values used for PR time for Case III ISD applications should be retained because of their validation from several empirical studies and the insensitivity in changes in ISD values relative to changes in PR times.
- A greater use of sensitivity analyses is encouraged, including the use of elasticities as a measure of effectiveness, to assess the relative importance of various design and operational parameters (e.g., highway capacity).

ACKNOWLEDGMENTS

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Transport of Manufactured Housing Units: Differential Effects of 4.27-m (14-ft)-Wide and 4.88-m (16-ft)-Wide Units on Traffic

FREDRICK M. STREFF, CHARLES C. MACADAM, AND LISA J. MOLNAR

The results of a study on the impact of transporting 4.27-m (14-ft)-wide versus 4.88-m (16-ft)-wide manufactured housing units on traffic operations in Michigan are reported. Observations of the home unit and other vehicles passing the home unit were made by observers traveling in a specially equipped vehicle and by observers subsequently reviewing videotapes recorded in the equipped chase vehicle. Measures of home unit encroachment onto the right shoulder and the passing lane or left shoulder were made in addition to estimates of home unit speed. Measures of use of the shoulder by vehicles passing the units were also made. In general more excursions from the normal travel lane were seen for the 4.88-m (16-ft)-wide units, and on average both 4.27-m (14-ft) and 4.88-m (16-ft) units traveled over the speed limit prescribed by the travel permits.

There has been growing concern about the increasing widths of manufactured housing units transported on U.S. roads and the impacts of these wider loads on the safety of other road users. In Michigan the transport of manufactured housing units was restricted to units less than approximately 4.27 m (14 ft) in width before 1991. In 1991 Michigan Senate Bill 142 authorized the transport of units up to 4.88 m (16 ft) in width for a period of 1 year, during which time the effects of the wider units on mobility and traffic operations were to be evaluated [the units being compared were 4.27 or 4.88 m (14 or 16 ft) wide and between 21.35 and 24.4 m (70 and 80 ft) long]. This paper reports findings from that evaluation conducted by the University of Michigan Transportation Research Institute (UMTRI).

Previous studies on the safety effects of transporting manufactured housing units have been scarce and have focused exclusively on widths of less than 4.88 m (16 ft). Parker et al. (*1*) reviewed the literature on 4.27-m (14-ft)-wide manufactured housing units and concluded that findings on the movement of such units were generally inconclusive because of small sample sizes or study methods used. Results from the authors' own evaluation of 3.66-m (12-ft)-wide and 4.27-m (14-ft)-wide units indicated no major differential effects on the safety and convenience of other road users. The authors found no statistically significant differences between unit widths in average speed, delay to traffic, vehicle passing time, or crash potential (as measured by a traffic conflicts technique). They did, however, find statistically significant differences in vehicle displacement and encroachment because of narrow structures and narrow pavements.

More recently Stoke (*2*) analyzed centerline and edgeline encroachment of standard 4.27-m (14-ft)-wide manufactured housing units and 4.27-m (14-ft)-wide experiment units with different 0.3-m (1-ft) roof-eave configurations. Stoke concluded that 4.27-m (14-ft)-wide manufactured housing units with eaves of up to an additional 0.3 m (1 ft) posed minimal additional safety risks to traffic on roads with four or more lanes but had the potential to pose additional safety risks to traffic on roads with two or three lanes.

Harkey et al. (*3*) also examined the differential effects of width on traffic operations and safety, but their focus was heavy trucks rather than manufactured housing units. The authors compared 2.59-m (102-in.)-wide and 2.44-m (96-in.)-wide trucks using videotape and slides. They found significantly higher rates of edgeline encroachment among the wider trucks than the narrower trucks. The wider trucks also tended to drive closer to the centerline than the narrower trucks. The authors cautioned, however, against generalizing their findings beyond rural two-lane highways and to trucks longer and wider than those in the study.

The study reported here focused on the differential effects of 4.27-m (14-ft)-wide and 4.88-m (16-ft)-wide [21.35 to 24.4 m (70 to 80 ft) long] manufactured housing units on maneuverability and adjoining traffic. Field data were collected in October and November 1991 to evaluate driver behavior in the presence of manufactured housing units in Michigan. Computer analysis was used to evaluate the low-speed maneuverability of the units as well as their highway-speed dynamic characteristics. Findings from the field study are summarized here. Readers interested in more detail on the field study as well as findings from the computer analyses are referred to the full report by MacAdam et al. (*4*).

METHODS

The field study was designed to gather data on both manufactured housing units and the vehicles passing them. Of interest were the differential effects of 4.27-m (14-ft)-wide and 4.88-m (16-ft)-wide units on the behavior of the units themselves as well as the behavior of passing vehicles. Because manufactured housing units of both 4.27-m (14-ft) and 4.88-m (16-ft) widths require a towing tractor during transport, the unit of interest in the field study was the entire tractor-home unit rather than just the home unit. Therefore the term *tractor-home unit* (or in some cases just *unit*) is used throughout the remainder of the paper to describe the manufactured housing unit and towing tractor being observed.

General Data Collection Protocols

In brief a vehicle equipped with a videotape unit followed behind the escort vehicle following the tractor-home unit. The videotape

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equipment in the observation vehicle generated a complete video record of each home delivery observed. In addition to the videotape record observers in the observation vehicle recorded behaviors of the tractor-home unit (i.e., lane encroachment) and vehicles passing the tractor-home unit (i.e. shoulder use) during the portion of the trip on multilane divided highways. Videotape and observation data were collected for a total of six deliveries of 4.27-m (14-ft)-wide units and seven 4.88-m (16-ft)-wide units.

Two identically configured vehicles were used for observations. Each data collection trip began with the observation vehicle traveling to the same rest area (located north of the Michigan-Indiana border). Observers waited there until a tractor-home unit of appropriate size [i.e., 4.27 or 4.88 m (14-ft or 16-ft) in width and 21.35 to 24.4 m (70 to 80 ft) in length] was seen approaching from the south. Once the tractor-home unit was observed approaching, the observers positioned their vehicle behind the escort vehicle following the tractor-home unit, started the video camera recording unit, and began recording background data about the route and the tractor-home unit.

The video camera was positioned in the camera mount so the view in the video monitor was filled by the road and the rear of the tractor-home unit. The field of view extended from the outside of the left shoulder to the outside of the right shoulder, with the camera lens focused at infinity. In addition to the view of the road and the tractor-home unit, the videotape was coded with the time the observation was made (hour, minute, and second of real time). This time stamp allowed linkages between the data recorded on the observation data sheets and the videotape record of the trip.

Analysis of tractor-home unit and passing vehicle behavior were conducted in two stages in the field study. First-stage analyses were based primarily on data recorded directly in the field, whereas second-stage analyses were based solely on review of the videotape logs made during field observations. In some cases the same behavior was analyzed in both stages (e.g., tractor-home unit encroachment into passing lane). In these cases the first-stage and second-stage analyses differed in terms of how the behavior was measured, the conditions under which it was measured, or the extent to which potentially confounding variables were examined. To enable readers to more easily compare results for similar behaviors, this paper is organized by topical area rather than sequential order of the analyses. Specific data collection protocols for each behavior observed are described briefly.

Encroachment by Tractor-Home Units

The primary goal of this portion of the study was to determine whether 4.88-m (16-ft)-wide tractor-home units encroached into the passing lanes of roads more than 4.27-m (14-ft)-wide units did. Passing lane refers to the lane to the left of the lane occupied by the tractor-home unit (i.e., the left adjoining lane). This lane is described as the passing lane throughout the paper, regardless of whether lane encroachment by the tractor-home unit occurred in the presence of a passing vehicle (overtaking the unit from the same direction) or an oncoming vehicle (overtaking the unit from the opposite direction).

Encroachment on Multilane Divided Highways

Encroachment of tractor-home units into the passing lane on multilane divided highways was examined in both first-stage and second-stage analyses by using different methods. During the first stage of the analyses, the encroachment time of the tractor-home unit was measured directly by observers in the field by using a timing apparatus mounted on the dashboard of the observation vehicle. Encroachment of the tractor-home unit was recorded only when a vehicle or platoon of vehicles began to pass the unit. This procedure was used because tractor-home unit encroachment is of little safety consequence unless vehicles are attempting to pass. Encroachment was measured in discrete events. An event was considered to be the period of time a vehicle or platoon of vehicles traveled from the front of the observation vehicle (passing maneuver initiation) to the front of the towing tractor (passing maneuver end).

During the second stage of the analyses the encroachment time of the tractor-home unit into the passing lane was based on review of the videotape logs made during the field observations. Data from the videotape logs were studied for each passing event. The encroachment time of the tractor-home unit into the passing lane was measured by using a computer program written especially for observers reviewing the videotape logs. The program was designed to measure two separate dimensions of encroachment behavior: tractor-home unit encroachment into the passing lane and tractor-home unit use of the shoulder.

Encroachment on Multilane Divided Highways and Two-Lane Roads by Lane Width, Shoulder Condition, and Road Type

Tractor-home unit encroachment was examined by lane width, shoulder condition (of the shoulder adjacent to the unit), and road type to determine whether factors other than unit width were related to encroachment. Both encroachment into the passing lane and use of the shoulder by the unit were examined because each represents a dimension of encroachment behavior. Encroachment time was calculated for both multilane divided highways and two-lane undivided roads on the basis of a review of the videotape logs in conjunction with the computer program discussed previously. Encroachment on two-lane undivided roads was limited to passing events by oncoming vehicles (although there were a few cases in which a vehicle overtook the unit while traveling in the same direction). Lane width, shoulder condition, and road type were determined on the basis of the observed road characteristics and information provided by the 1990 *Sufficiency Rating, Michigan State Trunkline Highways* (*5*).

Shoulder Use by Passing Vehicles

The goal of this portion of the study was to determine whether vehicles passing 4.88-m (16-ft)-wide tractor-home units used the shoulder of the road during the passing maneuver more often than vehicles passing 4.27-m (14-ft)-wide units. Data were collected for passing vehicles on multilane divided highways and for oncoming vehicles on two-lane undivided roads. Passing events on multilane divided highways included those in which vehicles were traveling in the same direction as the tractor-home unit and in the process of overtaking the unit. Passing events on two-lane undivided roads included those in which vehicles were passing the tractor-home unit in the oncoming direction. There were a few cases in which a vehicle traveling in the same direction as the tractor-home unit on a two-lane undivided road attempted to overtake the unit—these cases were also recorded and are discussed separately in the results.

Shoulder Use on Multilane Divided Highways and Two-Lane Undivided Roads

Shoulder use of passing vehicles on multilane divided highways was measured directly by observers in the field. Observations were made by the driver or observer seated in the front passenger seat of the observation vehicle. A vehicle was targeted for observation when it pulled even (in the passing lane) with the front of the observation vehicle. For each vehicle (or the first vehicle in a platoon of passing vehicles), the observer recorded the time from the video camera monitor.

Shoulder use of vehicles approaching the tractor-home unit in the oncoming lane on two-lane undivided roads was measured by observers reviewing the videotape logs of the trips. A vehicle was targeted for observation when it pulled even (in the oncoming lane) with the front of the tractor-home unit.

Shoulder Use on Multilane Divided Highways and Two-Lane Undivided Roads by Lane Width, Shoulder Condition, and Road Type

Shoulder use by passing and oncoming vehicles was examined by lane width, shoulder condition (of the shoulder adjacent to the passing vehicle), and road type to determine whether factors other than unit width were related to shoulder use. Shoulder use was observed for both passenger cars and heavy trucks (tractor semitrailers and doubles). Shoulder use was calculated for both multilane divided highways and two-lane undivided roads on the basis of a review of the videotape logs. Lane width, shoulder condition, and road type were determined on the basis of the observed road characteristics and information provided by the 1990 *Sufficiency Rating, Michigan State Trunkline Highways* (*5*).

Speed of Tractor-Home Units

The goal of this portion of the study was to examine the speeds of each of the tractor-home units during the trip. Once the observation vehicle caught up with the tractor-home unit and achieved a steady speed, the passenger seat observer queried the observation vehicle driver to determine the speed at which the vehicle was traveling. The driver reported the speed from the observation vehicle speedometer to the nearest 5-mph level. The passenger seat observer then held a prepared flash card up in front of the video camera to record the speed. This query and record system was repeated every 5 min throughout the trip. The speed data were transcribed from the videotape later by another observer who recorded speed of travel (from the flash card) and road type.

RESULTS

Encroachment of Tractor-Home Units

Encroachment on Multilane Divided Highways

Tractor-home unit encroachment into the passing lane on multilane divided highways was calculated by using two approaches. First the average of the proportion of time the units encroached into the passing lane during each passing event was calculated. That is, on the basis of the data recorded in the field, the total event time (from the stopwatch) was divided by the encroachment time from the timer, yielding a calculation of the proportion of time the tractor-home unit encroached during each event. This calculation resulted in events of different duration receiving an equal weight in the average encroachment time (e.g., a given event of 130 sec in duration in which the tractor-home unit encroached into the passing lane 40 percent of the time was given the same weight in the encroachment average as an event of only 30 sec in duration).

Because of concerns that differences in encroachment behavior that were potentially moderated by event duration would be overlooked by using the event-based encroachment average, a second approach was devised to estimate tractor-home unit encroachment into the passing lane that allowed all passing events to be given equal weight in proportion to their durations. On the basis of a review of the videotape logs, the second approach involved calculating the proportion of time the tractor-home units were observed to be encroaching into the passing lane by taking the total time that a specific unit encroached into the passing lane during all passing events and dividing this sum by the total time of all passing events for that unit.

Calculation 1 Results from the first calculation (the event-based encroachment average) indicated that, on average, 4.88-m (16-ft)-wide tractor-home units encroached into the passing lane during passing events on multilane divided highways more than 4.27-m (14-ft)-wide units. Specifically, 4.88-m (16-ft)-wide units were observed encroaching an average of 40.3 percent of the time for each passing event (168 total passing events), whereas 4.27-m (14-ft)-wide units were observed encroaching an average of 20.5 percent of the time for each passing event (128 total passing events).

There was a good deal of variation, however, between the encroachment behaviors of individual tractor-home units. That is, some tractor-home unit drivers encroached into the passing lane significantly less than other drivers. Average encroachment (over an entire delivery trip when adjoining traffic was present) for 4.88-m (16-ft)-wide units ranged from 3.4 to 60.9 percent. Average encroachment (over an entire delivery trip) for 4.27-m (14-ft)-wide units ranged from 2.3 to 54.3 percent.

When the entire range of encroachment time proportions was examined, it was found that 4.27-m (14-ft)-wide units did not encroach into the passing lane in 40 percent of all passing events, but 4.88-m (16-ft)-wide units did not encroach into the passing lane in only 10 percent of all passing events (Figure 1). This reinforces the finding that 4.88-m (16-ft)-wide units encroached into the passing lane more than 4.27-m (14-ft)-wide units.

Calculation 2 Results from the second calculation (based on the total encroachment time divided by the total event time over all events for each home) indicated that 4.88-m (16-ft)-wide units encroached into the passing lane on multilane divided highways more often than 4.27-m (14-ft)-wide units (43.9 versus 31.0 percent, respectively; Figure 1). These results are consistent with results from the first calculation of encroachment, although the absolute proportions differ.

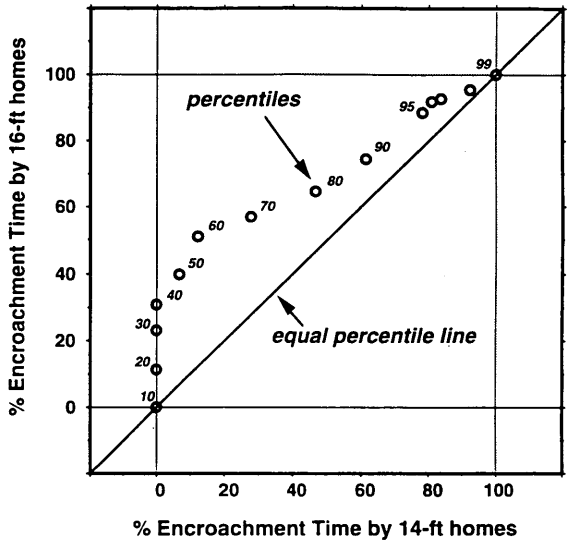

FIGURE 1 Percentile comparison of encroachment times for 4.88-m (16-ft) versus 4.27-m (14-ft) tractor-home units

Encroachment on Multilane Divided Highways and Two-Lane Undivided Roads by Lane Width, Shoulder Condition, and Road Type

Analyses of tractor-home unit encroachment on multilane divided highways and two-lane undivided roads by lane width, shoulder condition (of the shoulder adjacent to the unit), and road type were based on review of the videotape logs by using the second calculation of encroachment. Included were both tractor-home unit encroachment into the passing lane and tractor-home unit use of the shoulder. Because of the relatively small number of units observed, inferential statistics were not applied to the data. Instead a case study approach was used so that relationships could be examined as a whole, without having to interpret statistical values based on tests without sufficient statistical power to be meaningful because of small sample sizes. Readers should exercise caution in interpreting results; apparent differences may be the result of case-specific factors unrelated to the larger, more general population of vehicles.

Results for encroachment of tractor-home units by lane width, shoulder condition, and road type are presented in Table 1. Units of both widths were more likely to encroach into the passing lane on roads with 3.35-m (11-ft) lanes that on roads with 3.66-m (12-ft) lanes. However lane width appeared to have little effect on shoulder use for all units combined. On roads with 3.35-m (11-ft)-wide lanes, encroachment of 4.27-m (14-ft)-wide units into the passing lane was greater than that of 4.88-m (16-ft)-wide units. The reverse was true on roads with 3.66-m (12-ft)-wide lanes. Shoulder use, however, was greater among 4.88-m (16-ft)-wide units than 4.27-m (14-ft)-wide units on all roads, regardless of lane width.

Tractor-home units of both widths were more likely to encroach into the passing lane on road segments with no appreciable shoulder (e.g., no paved or improved shoulder) than on road segments with the shoulder in good condition. The reverse pattern was found for shoulder use. That is, units of both widths were more likely to use the shoulder when it was in good condition than when there was no appreciable shoulder. On road segments with a good shoulder, 4.88-m (16-ft)-wide units were more likely than 4.27-m (14-ft)-wide units to encroach into the passing lane. On road segments with no appreciable shoulder, the reverse was true. Shoulder use was greater among 4.88-m (16-ft)-wide units than 4.27-m (14-ft)-wide units both on roads with a good shoulder and on roads with no appreciable shoulder.

Tractor-home units of both widths were more likely to encroach into the passing lane on two-lane roads than on multilane divided highways. Although this may be due in part to the characteristics of the shoulder, this hypothesis could not be adequately explored because of the lack of sufficient data on the various shoulder characteristics for the different roads. There was essentially no difference in the overall use of the shoulder between two-lane and multilane roads. Although 4.88-m (16-ft)-wide units were more likely than 4.27-m (14-ft)-wide units to encroach into the passing lane on multilane divided highways, there appeared to be little difference between the two types of units on two-lane roads. On both multilane divided and two-lane roads, 4.88-m (16-ft)-wide units were more likely than 4.27-m (14-ft)-wide units to use the shoulder.

Shoulder Use by Passing Vehicles

Shoulder Use on Multilane Divided Highways and Two-Lane Undivided Roads

Figure 2 gives shoulder use by passing vehicles on multilane divided highways and oncoming vehicles on two-lane undivided

TABLE 1 Tractor-Home Unit Encroachment into Passing Lane and Use of Shoulder by Lane Width, Shoulder Condition, and Road Type

	4.26-Meter Wide[2]		4.88-Meter Wide		Both Widths	
	Encroachment % (N)	Shoulder Use % (N)	Encroachment % (N)	Shoulder Use % (N)	Encroachment % (N)	Shoulder Use % (N)
Lane Width						
3.35-Meter	46.8 (3)	27.3 (3)	29.5 (5)	91.2 (5)	36.2 (8)	67.2 (8)
3.66-Meter	4.9 (6)	54.8 (6)	15.5 (7)	79.8 (7)	10.7 (13)	68.3 (13)
Shoulder Condition						
Good	4.8 (6)	55.3 (6)	16.8 (7)	80.8 (7)	11.3 (13)	69.0 (13)
No Appreciable	50.0 (2)	20.4 (2)	40.7 (3)	58.0 (3)	44.4 (5)	43.0 (5)
Road Type						
Multilane Divided	5.3 (6)	75.8 (6)	15.2 (7)	80.7 (7)	10.7 (13)	70.2 (13)
Two-Lane	36.3 (2)	37.1 (2)	30.3 (5)	81.5 (5)	32.0 (7)	68.8 (7)

[2]1 meter = 3.28 feet

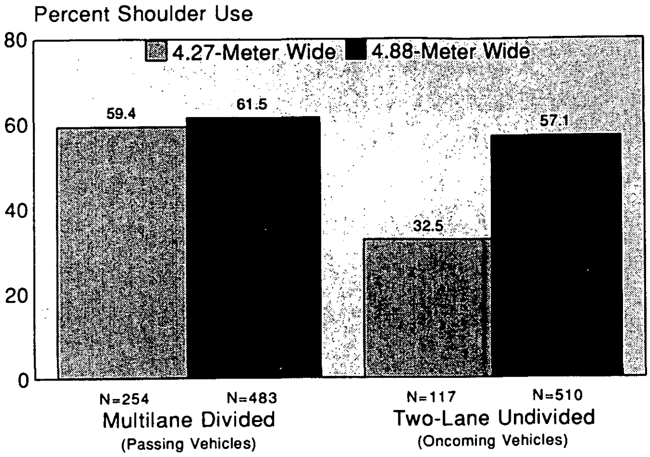

FIGURE 2 Shoulder use on multilane divided highways and two-lane undivided roads (1 m = 3.28 ft).

roads. On multilane divided highways a majority of passing vehicles used the shoulder when passing both 4.27-m (14-ft)-wide and 4.88-m (16-ft)-wide units. However few apparent differences between 4.27-m (14-ft)-wide and 4.88-m (16-ft)-wide units in the shoulder use of passing vehicles were found. On two-lane undivided roads a majority of passing vehicles used the shoulder only when passing 4.88-m (16-ft)-wide units.

Shoulder Use on Multilane Divided Highways and Two-Lane Undivided Roads by Lane Width, Shoulder Condition, and Road Type

Analyses of shoulder use by lane width, shoulder condition (of the shoulder adjacent to passing vehicle), and road type were based on review of the videotape logs and included shoulder use of both passenger vehicles and heavy trucks. As in the case of the supplemental analyses of tractor-home unit encroachment, a case study approach was used because of limited sample sizes. Readers should exercise caution in interpreting results; apparent differences may be the result of case-specific factors unrelated to the larger, more general population of vehicles.

Overall both cars and trucks were more likely to use the shoulder when passing 4.88-m (16-ft)-wide units than when passing 4.27-m (14-ft)-wide units, and trucks were more likely than cars to use the shoulder [15.6 percent (n = 960) of cars versus 35.7 percent (n = 140) of trucks used the shoulder when passing 4.27-m (14-ft)-wide units; 28.0 percent (n = 1,462) of cars versus 62.6 percent (n = 131) of trucks used the shoulder when passing 4.88-m (16/ft)-wide units]. Results for shoulder use of passenger cars and heavy trucks by lane width, shoulder condition, and road type are presented in Table 2.

Shoulder use by cars was greater on roads with 3.35-m (11-ft)-wide lanes than 3.66-m (12-ft)-wide lanes when passing both 4.88-m (16-ft)-wide and 4.27-m (14-ft)-wide units. On roads with 3.66-m (12-ft)-wide lanes, cars were more likely to use the shoulder when passing 4.88-m (16-ft)-wide units than when passing 4.27-m (14-ft)-wide units, whereas on roads with 3.35-m (11-ft)-wide lanes, there was little difference in shoulder use between cars passing 4.27-m (14-ft)-wide and 4.88-m (16-ft)-wide units. Analyses of truck shoulder use were limited to roads with 3.66-m (12-ft)-wide lanes because of insufficient cases of trucks passing on roads with other lane widths. On roads with 3.66-m (12-ft)-wide lanes, trucks used the shoulder more often when passing 4.88-m (16-ft)-wide units than when passing 4.27-m (14-ft)-wide units.

Cars were more likely to use the shoulder when passing on roads with a good shoulder and less likely to use the shoulder when passing on roads with no shoulder. Analyses of truck shoulder use were limited to roads with a shoulder classified as "OK" because of insufficient cases of trucks passing on roads with other shoulder classifications. Regardless of shoulder condition, both cars and trucks were more likely to use the shoulder when passing 4.88-m (16-ft)-wide units than when passing 4.27-m (14-ft)-wide units, except in the case of cars passing on roads with a good shoulder (note, however, the small sample size).

Both cars and trucks were more likely to use the shoulder when approaching tractor-home units in the oncoming direction on two-lane roads than when passing on multilane divided highways. As was the case in the analyses of tractor-home unit encroachment, these findings may be due in part to characteristics of the shoulder. This hypothesis could not be adequately explored because of the lack of sufficient data on the various shoulder characteristics for the different roads. Cars were more likely to use the shoulder when approaching 4.88-m (16-ft)-wide units in the oncoming direction on two-lane roads, but their shoulder use was nearly the same as that of trucks when passing units of different widths on multilane divided highways. Trucks on the other hand were more likely to use the shoulder when passing 4.88-m (16-ft)-wide units than when passing 4.27-m (14-ft)-wide units, regardless of whether they were traveling on two-lane undivided roads or multilane divided highways.

On rare occasions vehicles traveling in the same direction as the tractor-home unit on two-lane undivided roads passed the unit. In such cases when an oncoming vehicle was also present, the oncoming vehicle was forced completely onto the shoulder of the road to avoid a collision with the passing vehicle.

Speed of Tractor-Home Units

Results from the speed observations are given in Figure 3. The speed limit for such vehicles is 72.5 km/hr (45 mph) on highways

TABLE 2 Shoulder Use by Passing Passenger Cars and Heavy Trucks by Lane Width, Shoulder Condition, and Road Type

	4.27-Meter Wide		4.88-Meter Wide		Both Widths	
	Passenger Cars % (N)	Heavy Trucks % (N)	Passenger Cars % (N)	Heavy Trucks % (N)	Passenger Cars % (N)	Heavy Trucks % (N)
Lane Width						
3.35-Meter	51.2 (41)	----	49.3 (322)	----	49.6 (363)	----
3.66-Meter	14.0 (919)	35.7 (140)	22.2 (1127)	57.0 (107)	18.5 (2046)	44.9 (247)
Shoulder Condition						
None	1.6 (124)	----	9.0 (166)	----	5.9 (290)	----
OK	15.5 (802)	38.6 (127)	30.0 (1261)	63.9 (122)	24.3 (2063)	51.0 (249)
Good	70.6 (34)	----	45.7 (35)		58.0 (69)	
Road Type						
Multilane Divided	17.2 (692)	37.7 (114)	18.5 (905)	54.8 (93)	17.9 (1597)	45.4 (207)
Two-Lane	23.3 (120)	55.6 (9)	50.9 (458)	88.2 (34)	45.2 (578)	81.4 (43)

with four or more lanes and 56.4 km/hr (35 mph) on highways with fewer than four lanes. As shown in Figure 3, vehicles of both widths consistently drove in excess of the speed limit prescribed on their travel permits. There was no difference between the average speeds of 4.27-m (14-ft)-wide units and 4.88-m (16-ft)-wide units.

SUMMARY AND CONCLUSIONS

Overall results of the field study are summarized. With regard to encroachment of tractor-home units, it was found that 4.88-m (16-ft)-wide units were more likely than 4.27-m (14-ft)-wide units to encroach into the passing lane while they were being passed by other vehicles on multilane divided highways. Although these encroachments degrade the level of safety on these roads, the level and effect of this degradation are unclear. To assess the significance of the effect of these encroachments on safety, the behavior of drivers attempting to pass the tractor-home units was examined.

One might expect that passing vehicles would be forced onto the shoulder of the road more often by the 4.88-m (16-ft)-wide units because these units were more likely to encroach into the passing lane. However no relationship was found between the shoulder use behavior of passing vehicles on multilane divided highways and the width of the tractor-home unit being passed. This finding complicates the question of the safety impact of 4.88-m (16-ft)-wide units. That is, although intuitively it would seem that if tractor-home units encroached more into other lanes, there would be a detrimental effect on the ability (or desire) of passing vehicles to remain in their lanes, this was not found to be the case. In fact passing vehicles on multilane divided highways were found to use the shoulder nearly two-thirds of the time, regardless of the width of the tractor-home unit being passed. This finding does not support the contention that 4.88-m (16-ft)-wide units degrade the safety of drivers traveling around those units more than 4.27-m (14-ft)-wide units. However these findings do suggest that both 4.27-m (14-ft)-wide and 4.88-m (16-ft)-wide units degrade the safety of vehicles trying to pass those units. This degradation of safety is based on the fact that vehicle drivers used the shoulder rather than the travel lanes to complete passing maneuvers. Use of the shoulder decreased the margin of error for road departure available to vehicles passing the units. In addition the conditions of shoulder surfaces are often much poorer than those of normal travel lanes, thereby increasing the chances of vehicle control problems for vehicles that use the shoulder.

The shoulder use behavior of oncoming vehicles on two-lane undivided roads differed somewhat from that of passing vehicles on multilane divided highways. That is, no difference in shoulder use was found for vehicles passing 4.27-m (14-ft)-wide and 4.88-m (16-ft)-wide units on multilane divided highways [although a majority of drivers passing both 4.27-m (14-ft)-wide and 4.88-m (16-ft)-wide units used the shoulder], but a noticeable difference in shoulder use was found between vehicles approaching 4.27-m (14-ft)-wide and 4.88-m (16-ft)-wide units in the oncoming lane on two-lane undivided roads. Drivers passing an oncoming 4.88-m (16 ft)-wide unit were more likely than drivers passing an oncoming 4.27-m (14-ft)-wide unit to use the shoulder. In fact although 57 percent of drivers used the shoulder when passing an oncoming 4.88-m (16-ft)-wide unit, only 32 percent of drivers used the shoulder when passing an oncoming 4.27-m (14-ft)-wide unit.

What is clear is that the shoulder use of vehicles on two-lane undivided roads represents a reduction in safety. In many of the shoulder use events on two-lane undivided roads the observed drivers chose to move off of the paved road surface onto an unpaved shoulder area. The drop-off from and return to a paved road surface is a potentially hazardous vehicle maneuver that should generally be avoided because it can lead to loss of control. In addition driving on an unpaved surface is generally more hazardous than driving on a paved surface because of reduced tire friction and the uneven surface. This type of behavior by passing drivers was far less frequent on multilane divided highways.

With regard to tractor-home unit speeds, it was found that tractor-home units of both widths regularly traveled in excess of the maximum speed specified on their travel permits. Units of 4.88 m (16 ft) in width traveled at almost the same average speeds as units of 4.27 m (14 ft) in width; however, the effects of this speeding behavior on safety may differ between the units. The specific effects of this speeding behavior on the safety of the tractor-home units and on the traffic that must interact with the units are unclear. Some may argue that the higher tractor-home unit speeds simply act to reduce the speed variance on the road and thus actually improve safety. On the other hand these units are in clear violation of the lawfully permitted speed. In addition the dynamics of tractor-home unit stability is affected negatively by the higher observed, travel speeds, and the stopping distance requirements are

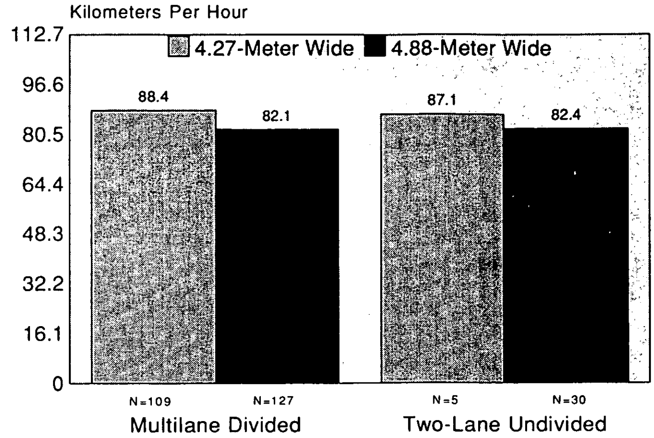

FIGURE 3 Speed of tractor-home units (kph = mph × 1.61; 1 m = 3.28 ft).

significantly increased because of the poor braking performance of tractor-home units.

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