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Contents

Foreword

This Record compiles current information on intersections and interchanges. The first 7 papers are from the 1991 Annual Meeting, the next 13 are from the 1992 Annual Meeting, and the final paper was presented at the 1993 Annual Meeting.

Fitzpatrick et al. compare intersection sight distances from models in the AASHTO Green Book for turning vehicles. Note that NCHRP Project 15-14(1), Intersection Sight Distance, also addresses this and other intersection sight distance issues. Pietrucha and Opiela examine the highway and street intersection design process from a pedestrian's perspective. Stover relates access control issues with a generalized intersection hierarchy derived from functional design aspects of the AASHTO Green Book. Mason et al. present an alternative perspective on design of intersections that accommodate the turning radii of large trucks. Bonneson et al. outline current practices of state highway departments toward mitigation of accidents at intersections on rural expressways. Improvements presented are categorized as either traffic control measures or geometric design measures. Leisch discusses and presents various forms of grade-separated intersections to increase capacity (especially of through movements) or resolve physical constraints of the intersecting roadways. Walker promotes proper coordination of the various elements (horizontal and vertical alignments, drainage, etc.) that must be considered in intersection design with driver and pedestrian decisions before and during negotiation of the intersection.

Leisch provides background on which interchange planning and design are based. Lamm et al. present an international view of interchanges. The papers by Lunenfeld, Holzmann and Marek, Twomey et al., Leisch, and Merritt add to the knowledge base on interchanges, concentrating on human factors, the interchange selection process, accidents and safety, operational considerations, and specifics of single-point urban interchange design.

Harwood and Mason focus on off-ramps, promoting increased attention to the difference between mainline speed and the design speed of the ramp. Koepke deals with geometrics of acceleration and deceleration lanes: tapered versus parallel design. Walker presents operational, safety, and capacity aspects of two-lane loop ramps. Geometric design standards, current practices, signing, marking, and other aspects relating to metered entrance ramps and high-occupancy vehicle bypass lanes are presented by Lomax and Fuhs. Keller discusses various aspects that state highway agencies consider when designing the alignment and superelevation of interchange ramps. Plummer et al. focus on at-grade ramp terminals, emphasizing the appropriateness of geometric design, operation, and capacity aspects of intersections.

Kikuchi et al. present capacity and other considerations in determining lengths of left-turn lanes at signalized intersections.

Comparison of Sight Distance Procedures for Turning Vehicles from a Stop-Controlled Approach

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

The AASHTO Green Book intersection sight distance procedures for turning vehicles are based on the passenger car as the design vehicle. Highway design and operational criteria, however, should consider the characteristics of vehicles that can be expected to use a facility with reasonable frequency. Equations were developed to reproduce the intersection sight distance values presented in the 1984 Green Book. By using different values for certain parameters (i.e., truck acceleration instead of passenger car acceleration information), sight distance values for other conditions can be calculated. When truck characteristics from the Green Book are used in the equations, values over ½ mi in length for certain design speeds result. Operational and safety experiences at intersections indicate that sight distances of this magnitude are not needed for safe and efficient operations. Procedures based on actual operations at an intersection should result in values that better reflect sight distances drivers use. An intersection sight distance procedure based on the gaps a driver safely accepts during actual intersection operations is presented. Field data on the various intersection sight distance parameters and gap acceptance data were obtained from studies at six intersections. These data are used to develop intersection sight distance values from (*a*) the proposed gap acceptance procedure and (*b*) current parameter values used in the developed equations. These results are compared with both the 1984 and 1990 Green Book intersection sight distance values.

The 1990 edition of AASHTO's *A Policy on Geometric Design of Highways and Streets* (*1*) (known as the Green Book and as GB90 as used herein) contains stop-controlled intersection sight distance (ISD) procedures. The procedures for turning vehicles are based on consideration of the passenger car as the design vehicle. Highway design and operational criteria, however, should consider the characteristics of vehicles that currently use or anticipate using a facility with reasonable frequency.

This paper briefly reviews both the 1990 (GB90) and 1984 (GB84) AASHTO ISD procedures (*1,2*). Truck parameter values derived from AASHTO information are used in recently developed equations to illustrate the need to consider alternative procedures. One alternative procedure is to base ISD on the gaps that drivers typically accept during actual intersection operations. Data collected at several intersections are used in the "developed equations" and in the proposed gap acceptance procedure to produce values based on current vehicle performance. These values are compared with GB90 and GB84 ISD values.

This paper synthesizes several previous publications on intersection sight distance. Additional information can be obtained from several detailed publications including the Harwood et al. study on truck characteristics (*3*) and the work by Fitzpatrick on passenger cars (*4*). Previous Transportation Research Board publications report on the sensitivity analyses (*5*), the field studies (*6*), the reviews of ISD Case III (*7*) and ramp terminal (*8*) procedures, and the gaps accepted by truck and passenger car drivers (*9*).

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AASHTO INTERSECTION SIGHT DISTANCE PROCEDURES

Three basic maneuvers occur at a stop-controlled intersection: traveling across the intersecting road by clearing traffic on both the left and the right of the crossing vehicle, turning left onto the intersecting road by first clearing traffic on the left and then entering the traffic stream with vehicles from the right, and turning right onto the intersecting road by entering the traffic stream with vehicles from the left. Consequently, there are three separate sight distance criteria for a vehicle stopped at an intersection. These conditions are referred to by AASHTO as Cases IIIA, IIIB, and IIIC, respectively. Sight distance values are shown in GB84 Figure IX-27 and GB90 Figure IX-40.

Case IIIA—Crossing Maneuver

As stated in the GB90, "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." The sight distance may be calculated from an equation or by using the GB90 Figure IX-39.

Case IIIB—Turning Left onto a Crossroad

A vehicle turning left onto a crossroad should have sight distance to a vehicle approaching from either the right or the left (see Figure 1). In the GB90, the turning vehicle should

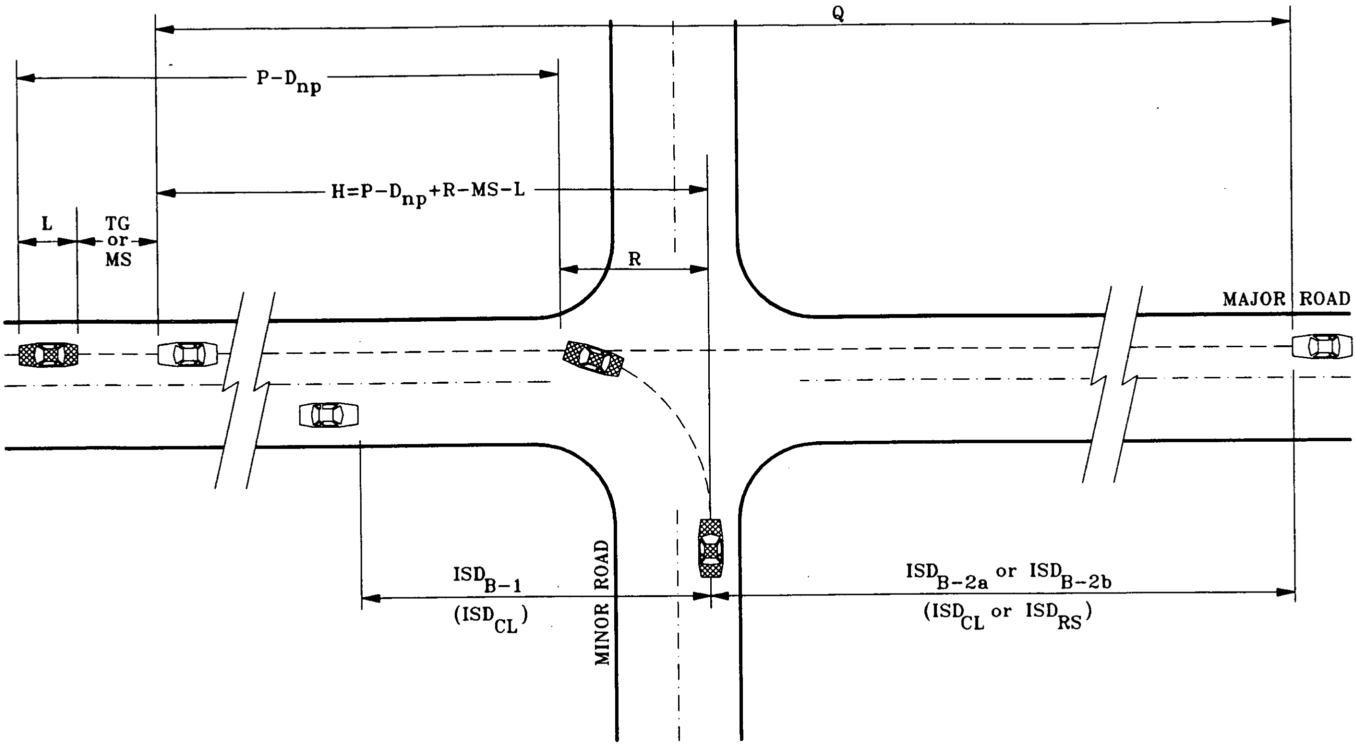

FIGURE 1 Distance considered in a left-turn maneuver.

be able to clear the near lane by the time the major-road vehicle approaching from the left arrives (ISD_{B-1}), and the turning vehicle should be able to accelerate to 85 percent of design speed by the time the major-road vehicle approaching from the right is within a specified vehicle gap distance while traveling at 85 percent of design speed (ISD_{B-2b}). (GB90 Figures IX-36 and IX-37 contain the details of the procedure.) In the GB84, values were provided for the turning vehicle to be able to accelerate to the average running speed by the time the approaching vehicle is within a certain tailgate distance after reducing its speed to the average running speed, or the turning vehicle is able to accelerate to the design speed by the time the approaching vehicle is within a certain tailgate distance maintaining the design speed. (Figure IX-24 in the GB84 contains the details of the procedure.) Distances to the left that provided time for the turning vehicle to clear the near lane were also provided in GB84 Figure IX-27.

In both Green Book versions, AASHTO states that the required sight distances for trucks turning left onto a crossroad will be substantially longer than those for passenger cars. AASHTO further indicates that the sight distance for trucks can be determined using appropriate assumptions for vehicle acceleration rates and turning paths. The specific assumptions, however, are not detailed in AASHTO policy.

Case IIIC—Turning Right onto a Crossroad

In the GB90, the turning vehicle should be able to accelerate to 85 percent of design speed by the time the major-road vehicle is within a specified vehicle gap distance while traveling at 85 percent of design speed. (Figure IX-38 in the GB90 provides the details of the procedure.) In the GB84, a right-turning vehicle should have sufficient sight distance to vehicles approaching from the left to complete its right turn and to accelerate to either design speed or average running speed before being overtaken by traffic approaching the intersection from the left traveling at design speed or reducing to average running speed. (The Case IIIC policy is described in Figure IX-25 in the GB84.) Similar to Case IIIB, AASHTO indicates that sight distances for trucks need to be considerably longer than for passenger vehicles.

REPRODUCTION OF GB84 SIGHT DISTANCE VALUES

GB84 Figure IX-27 contains six sight distance versus design speed curves. Two of these curves are the upper and lower limits for stopping sight distance (SSD). The curve labeled A represents Case IIIA methodology and is the sight distance for a passenger car crossing a two-lane highway from a stopped position. Derivations for the SSD and A curves are included in the GB84 (and in the GB90).

The assumptions and procedures to derive the remaining three curves were not included in the GB84. Some discussions of these curves are included in the GB90, but again no derivation is given. These curves represent the sight distance required for the following maneuvers:

- Left-turning vehicle to clear the near lane (Green Book Case IIIB, B-1 Curve or "Clear Lane," CL, procedure),
- Turning vehicle to accelerate to design speed while major-road vehicle maintains a constant speed (Green Book Case

IIIB and C, B-2a & Ca Curve or "Constant Speed," CS, procedure), and

- Turning vehicle to accelerate to running speed while major-road vehicle reduces speed from design speed to running speed (Green Book Case IIIB and C, B-2b & Cb Curve or "Reduce Speed," RS, procedure)

Sufficient information to easily identify some of the assumed parameter values was missing in the GB84. Using the information that was provided, and making some reasonable assumptions for the missing information, Fitzpatrick and Mason (*7*) reproduced these curves within 8 percent accuracy. Table 1 gives the sight distance values calculated using truck characteristics in the equations developed by Fitzpatrick and Mason (hereafter referred to as the "developed equations") and the sight distance values from both Green Book editions.

The equations to reproduce the Reduced Speed procedure (B-2b & Cb Curve) are given in Table 2. Figure 1 shows the parameters used in the equations. The derivation of the equations and the equations to reproduce the other sight distance values (Clear Lane and Constant Speed procedures) are presented elsewhere (*3,4,7*).

Truck characteristics used in the developed ISD equations frequently produced sight distance values greater than those at which drivers can normally detect motion. Tunnard and Pushkarev (*10*) state that an individual cannot perceive movement much beyond 800 ft or discern detail beyond 1,400 ft because the vehicle is too small and that a car at 2,000 ft appears the size of a pinhead held at 18 in. The Constant Speed procedure produced a sight distance for trucks of 3,200 ft for a 50-mph major-road design speed; the Reduce Speed procedure resulted in a sight distance for trucks of 2,500 ft for the same design speed. Operational experience at intersections indicates that sight distances of such magnitude are rarely necessary for safe and efficient operations.

Generally speaking, intersections currently operate with sight distance less than those calculated and, for practical reasons, ISD procedures should reflect actual field operations. For example, the individual parameter values used in the ISD procedures should represent current and/or future vehicle and driver characteristics. This can be accomplished by explicitly considering gaps in the major-road traffic that are accepted by minor-road drivers. Gap acceptance involves the evaluation of available gaps in an opposing traffic stream and the decision to carry out a specific maneuver within a particular gap. At a stop-controlled intersection, drivers observe the gaps in the traffic streams and then join or cross the major-road traffic stream within the length of the selected gap. Gap acceptance data are, therefore, suggested for use to determine the required sight distance at intersections.

EQUATIONS FOR GB90 SIGHT DISTANCE VALUES

The GB90 contains equations either within the Case IIIB and IIIC discussions or on the figures that describe the cases (see GB90 Figures IX-36, 37, and 38). These equations and a description for each parameter are given in Table 3. Where terms are similar to the previous set of developed equations (see Table 2) but have different values (for example, the new PC acceleration values), "1990" is added after the term to avoid confusion between the two sets of equations. When the GB90 equations were used, the authors of this paper calculated B-2 & Cb intersection sight distance values between 11 and 16 percent less than the values shown in GB90 Figure IX-40. Even though the GB90 indicates that the major-road vehicle decelerates during the minor-road vehicle's turning maneuver, no terms for time at design speed or time for deceleration are included in the GB90 equations (see Table 3). When these terms are considered (the equation for Q distance in Table 2 was used with appropriate GB90 parameter values), the calculated sight distances are 8 percent greater than the previous attempt at reproducing the GB90 values. Expressed in another manner, the calculated values that con-

TABLE 1 Intersection Sight Distance Values

Speed (mph)	Green Book Sight Distance Values (ft)					Calculated Sight Distance Values for Trucks[d] (ft)		
	B-1 Curve		B-2a & Ca Curve[a,c]	B-2b & Cb Curve		B-1-WB50 Clear Lane	BT-2a & Ca Constant Speed	BT-2b & Cb Reduced Speed
	GB84[a]	GB90[b]		GB84[a]	GB90[b]			
20	300	210	250	250	240	687	670	670
25	350	260	340	325	300	858	903	903
30	425	310	450	425	370	1,030	1,179	1,179
35	500	360	580	525	470	1,202	1,516	1,213
40	550	420	750	660	570	1,374	1,938	1,549
45	625	470	950	825	710	1,545	2,483	1,971
50	675	510	1,190	1,025	850	1,717	3,199	2,516
55	750	570	1,440	1,225	980	1,889	* [e]	3,232
60	825	620	1,730	1,475	1,150	2,060	*	*
65	875	670	2,100	1,725	1,350	2,232	*	*
70	950	710	2,500	2,000	1,550	2,404	*	*

[a] Values from GB84 Figure IX-27.
[b] Values from GB90 Figure IX-40.
[c] Concept is briefly discussed in the GB90, however, no values or procedures are provided.
[d] Calculated using equations developed to reproduce GB84 values [7]. Truck characteristics are based on GB84 information.
[e] Acceleration time and distance information not available.

TABLE 2 Equations Developed to Reproduce GB84 ISD Values

$ISD_{B\text{-}2b\ \&\ Cb}$ or $ISD_{RS} = Q - H$	$t_{rs} = t - t_{ds} - t_{dec}$	$t_{dec} = (2 * D_{dec}) / (1.47\ V_{ds} + 1.47\ V_{rs})$
$Q = 1.47\ V_{ds}\ t_{ds} + D_{dec} + 1.47\ V_{rs}\ t_{rs}$	$t = t_t + J$	$D_{np} = \pi * R/2$
$H = P - D_{np} + R - TG - L$	$t_{ds} = J + t_{pr}$	$TG = 1.47\ V_{rs}\ t_{TG}$

$ISD_{B\text{-}2b\ \&\ Cb}$ or ISD_{RS}	=	sight distance along the major roadway's far lane to the right for left turns and along the near lane to the left for right turns. This condition assumes that the major road vehicle reduces speed from design speed to running speed during minor road vehicle's turning maneuver (ft), see Figure 1
Q	=	distance traveled by the major road vehicle during the minor road vehicle's turning maneuver (ft)
V_{ds}	=	design speed of major road vehicle (mi/h)
t_{ds}	=	time major road vehicle is at design speed during turning maneuver (sec)
D_{dec}	=	distance traveled during deceleration (ft), data can be derived from GB84 Figure II-13 or GB90 Figure II-17
t_{dec}	=	time major road vehicle is decelerating (sec)
V_{rs}	=	running speed of major road vehicle (mi/h), assumed as design speed when design speed is 30 mi/h or less, as 5 mi/h below design speed for design speeds of 35 to 65 mi/h, or as 10 mi/h below design speed when design speed is 70 mi/h
t_{rs}	=	time major road vehicle is at running speed during turning maneuver (sec)
H	=	major road vehicle's distance from intersection when at assumed tailgate distance to minor road vehicle (ft)
t	=	time for a stopped minor road vehicle to move into traffic stream and accelerate to design speed (sec)
J	=	sum of perception time and time required to actuate the clutch or automatic shift (sec), assumed: $J = 2.0$ sec
t_t	=	acceleration time for the minor road vehicle to complete the turning maneuver (sec), data can be derived from GB84 Figure IX-22 or GB90 Figure IX-34 and IX-34A
t_{pr}	=	perception-reaction time for the major road driver (sec), assumed: $t_{pr} = 2.0$ sec
P	=	total distance traveled by minor road vehicle from stopped position to location when the reduced speed is achieved (ft), data derived from GB84 Figure IX-22 or GB90 Figure IX-34
D_{np}	=	distance minor road vehicle traveled during the turning maneuver that is not parallel to major highway (ft)
R	=	radius of turn for minor road vehicle (ft)
TG or MS	=	tailgate or minimum separation distance (ft)
L	=	length of minor road vehicle (ft)
t_{TG} or t_{MS}	=	tailgate or minimum separation time (sec), assumed: $t_{TG} = 1.0$ sec

TABLE 3 Equations for the GB90 ISD Procedure

$ISD_{B\text{-}2b\ \&\ Cb,1990} = Q_{1990} - h_{1990}$	$h_{1990} = P_{1990} - 16 - VG - L$
$Q_{1990} = 1.47\ V_{85\%ds,1990}\ (t_{1990} + J)$	$VG = 1.47 * V_{85\%ds,1990} * t_{VG}$

$ISD_{B\text{-}2b\ \&\ Cb,1990}$	=	sight distance along the major roadway's far lane to the right for left turns and along the near lane to the left for right turns (ft)
Q_{1990}	=	distance traveled by the major road vehicle during the minor road vehicle's turning maneuver (ft)
$V_{85\%ds,1990}$	=	85 percent of design speed of major road vehicle (mi/h)
h_{1990}	=	major road vehicle's distance from intersection when at assumed vehicle gap distance to minor road vehicle (ft)
J	=	sum of the perception time and the time required to actuate the clutch or automatic shift (sec), assumed: $J = 2.0$ sec
t_{1990}	=	acceleration time for the minor road vehicle to reach 85 percent of design speed from a stopped position (sec), data from GB90 Table IX-7
P_{1990}	=	total distance traveled by minor road vehicle from stopped position to location when 85 percent of design speed is achieved (ft), data from GB90 Figure IX-34
16	=	the distance (in ft) that the minor road vehicle traveled during the turning maneuver that is parallel to the major road, determined using the following equation: $\pi R/2 - R$ where R is the radius of turn for the minor road vehicle (ft)
VG	=	vehicle gap distance (ft)
L	=	length of minor road vehicle (ft), assumed: $L = 19$ ft
t_{VG}	=	vehicle gap time (sec), assumed: $t_{VG} = 2.0$ sec

sider deceleration are found to be between 3 and 10 percent less than the values in GB90 Figure IX-40.

The sight distance values in the GB90 are between 4 and 23 percent less than the values in the GB84 edition. Changes between the two Green Book editions in the Reduced Speed procedure include the following:

- The distance or time to accelerate by a minor-road passenger car in the GB90 has been updated. Figures showing truck acceleration characteristics did not change.
- The vehicle gap (previously known as "tailgate" or "minimum separation") time between the turning vehicle and the major-road vehicle is assumed to be 2 sec in the GB90; it was estimated as 1 sec in the GB84 developed equations.
- In the GB90, the major-road vehicle decelerates to (or travels at) 85 percent of design speed, whereas Fitzpatrick and Mason used a 0 mi/hr reduction for design speeds of 30 mi/hr or less, a 5 mi/hr reduction for design speeds between 35 and 65 mi/hr, and a 10 mi/hr reduction for design speeds of 70 mi/hr to reproduce the GB84 values.

- As discussed in the previous paragraph, it has not been determined whether the GB90 procedure implicitly accounts for the time that the major-road vehicle is at design speed or is decelerating.

The reduction in sight distance values between the GB84 and GB90 editions can be primarily attributed to updated "distance" and "time to accelerate" values.

SURVEY OF INTERSECTION SIGHT DISTANCE PROCEDURES

To measure the acceptance of the GB84 ISD procedures, a letter was sent to the highway agencies of 50 states, Puerto Rico, and the District of Columbia in October 1988 requesting a copy of the pertinent intersection sight distance section in their agency design manuals (*4*). The following summarizes the responses:

- Twenty agencies indicated that they apply the ISD policies as presented in the GB84.
- Fifteen states' design manuals included portions of the policy discussed in the GB84.
- Seven states did not respond to the request.
- Four states used variations or portions of the GB84 procedures.
- Three states use SSD as the minimum sight distance values and either Curve B-1 SD values, twice SSD values, or values developed from field studies conducted by the state as the desirable ISD values.
- Two states use a gap acceptance procedure (7.0 and 7.5 sec).
- One state's design manual said "the location of each approach should be reviewed to ensure that sight restrictions do not create a hazardous condition."

FIELD STUDIES

Details of the field studies for both cars and trucks were documented previously (*3,4,6*); the following discussion provides a synopsis of the analytical procedures and results.

Gap Acceptance

The logistic function was selected on the basis of a literature review of statistical methods to evaluate the gap data obtained in the field studies. Detailed discussions on previous gap acceptance studies, statistical procedures to evaluate data, and the determination of the 50 percent and 85 percent probability of accepting a gap is contained elsewhere (*9*). The findings from the field studies compared favorably with other studies. The generalized results used in this paper for passenger cars and trucks are as follows:

	Passenger Car (sec)	*Five-Axle Trucks (sec)*
50%	6.5	8.5
85%	8.25	10.0
85% (low volume)	10.50	15.0

Acceleration

The time between departure from the intersection and arrival at specific increment points for an accelerating vehicle was determined from videotaped field data. Time data at each increment point were averaged in each intersection/vehicle/maneuver combination file. Several regression analyses were performed on each data set and the best fit equation was selected. The regression equation coefficients and limits and the number of vehicles used in the average are given in Table 4. The basic form of the regression equation is

$$D = \beta_1 t_a + \beta_2 t_a^2 \tag{1}$$

TABLE 4 Acceleration Regression Equation Coefficients

Intersection Characteristics	Turn Maneuver	Vehicle Type	Data Sets	β_1	β_2	Max. Time (sec)*
Low-volume, Rural	right	3- & 4-axle	8	9.351	0.516	24.39
	left	3- & 4-axle	26	2.432	0.767	20.44
Low-volume, Rural (Truck Stop)	right	5-axle	44	6.366	0.311	26.55
High-volume, Urban	right	5-axle	41	1.523	0.967	23.37
	left	5-axle	4	8.726	0.638	23.03
	right	PC	75	7.755	1.801	14.62
	left	PC	47	8.150	1.708	15.27

*The limits of the regression equations are from 0 sec to the value listed in this column. The values are the maximum acceleration time from the intersection data.

where

β_1, β_2 = regression coefficients,
D = distance to accelerate (ft), and
t_a = time to accelerate (sec).

Speed reached at a specific time was calculated by taking the first derivative of the regression equation. For example, a right-turning passenger car at a high-volume, urban intersection would reach 25 mi/hr after accelerating for 8.05 sec, as shown in the following calculations:

Time-distance equation:

$$D = 7.755t_a + 1.801t_a^2 \quad (2)$$

First derivative:

$$\frac{dD}{dt} = V = 7.755 + (2 * 1.801 * t_a) \quad (3)$$

Speed reached (V) when t = 8.05 sec:

$$V = 7.755 + (2 * 1.801 * 8.05)$$
$$= 36.75 \text{ fps} = 25.0 \text{ mi/hr} \quad (4)$$

Deceleration

An average speed was estimated for an approaching major-road vehicle at each 100-ft increment. These average speeds were examined to identify where a maximum deceleration rate or speed reduction occurred. Vehicles were not considered in the analysis if they had less than a 5-mi/hr speed reduction through the observation area or if the data displayed erratic or extreme speed variations. The minimum value of 5 mi/hr for speed reduction was selected on the basis of the estimated accuracy of the reduced data.

Table 5 gives the 50th and 85th cumulative deceleration rates and speed reduction values occurring before the intersection for major-road vehicles. These values typically represent a 200- to 400-ft total deceleration distance ending 50 to 150 ft before the intersection. Table 5 also gives the speed reduction for each 5-mi/hr rounded initial speed increment.

Minimum Separation

Minimum separation is the shortest distance between the rear bumper of the turning vehicle and the front bumper of a vehicle approaching on the major roadway at any point during the turning maneuver. Minimum separation can be approximated by comparing the acceleration data for the minor-road vehicle with the deceleration data for the vehicle approaching on the major roadway. The minimum time (or distance) difference between estimated acceleration and deceleration curves was determined from plots of the respective data. The available information on minimum separation distances was very limited; nonetheless, an attempt was made to establish a probable range of values for right-turning vehicles.

This limited analysis indicated a minimum separation time value of approximately 1 sec for right-turning passenger cars and for the trucks turning onto a low-volume, rural road.

TABLE 5 Deceleration Rates and Speed Reductions for Major-Road Vehicles

Cumulative Deceleration Rates and Speed Reductions

Major Road Vehicle Reacting to			Deceleration Rate (mi/h/sec)		Speed Reduction (mi/h)	
Turn	Vehicle	Lane	50 percent	85 percent	50 percent	85 percent
Right	5-axle	onto	3.67	5.85	21.2	38.1
Right	PC	onto	2.62	3.75	12.3	16.2
Left	PC	cross	2.05	3.07	15.3	20.1
Left	PC	onto	1.48	2.36	8.3	13.1

Estimated Speed Reductions by Rounded Initial Speed

Rounded Initial Speed (mi/h)	Estimated Speed Reduction in Response to			
	Left-Turning Passenger Car		Right-Turning	
	Onto Lane (mi/h)	Cross Lane (mi/h)	Passenger Car (mi/h)	Truck (mi/h)
25	12*	**	**	**
30	**	**	**	**
35	14	7	**	15
40	16	10	11	15
45	18	13	11	20
50	20	13*	13	25
55	**	25	14	30*
60	**	**	16*	35
65	**	**	16*	35*
70	**	**	**	40*

*Value based on two or less observations.
**No data available.

Higher values were generally found for five-axle trucks at an intersection on a high-volume, urban road. Drivers appeared to select a larger separation distance between their vehicle and the turning vehicle if available, but accepted 1 sec or less on some occasions. (A 1-sec tailgate or minimum separation time represents a 15-ft minimum separation distance for each 10 mi/hr increment.)

INTERSECTION SIGHT DISTANCE PROCEDURES

Another objective of this paper is to compare the AASHTO ISD values with the results from (*a*) a gap acceptance procedure and (*b*) the field study findings used in the developed ISD equations. The field study findings included gaps accepted by the minor-road driver, minor-road vehicle acceleration, major-road vehicle deceleration and speed reduction, and minimum separation findings.

Gap Acceptance Procedure Results

Sight distance values for both left- and right-turning vehicles given in Table 6 were calculated on the basis of design speed and gap acceptance times of 7.0, 8.25, and 10.5 sec for passenger cars and 8.5, 10.0, and 15.0 sec for trucks.

Passenger Cars

The 7.0-sec gap was selected on the basis of GB84 and GB90 discussions of local roads, findings from the field study, and results of the agency survey. The GB90 states that a "minimum of 7 sec should be available to the driver of a passenger vehicle crossing the through lanes" on a local road or street and that the resulting "sight distance should be sufficient to permit a vehicle on the minor leg of the intersection to cross the travel way without requiring the approaching through-traffic to slow down" (*1*).

The 7.0-sec gap is also greater than the 50 percent probability of a passenger car driver accepting a gap for both left and right turns (6.50 sec). Two states also use a similar value in their gap procedure to calculate ISD; California uses 7.5 sec, and Michigan cites 7.0 sec. The 8.25-sec gap is the 85 percent probability of accepting a gap for both right- and left-turning passenger cars at moderate- to high-volume (intersection ADT between 10,000 and 22,000 vehicles) intersections. The 10.5-sec gap represents the 85 percent probability of accepting a gap at an intersection where the accepted gaps were influenced by low to moderate volumes (intersection ADT less than 10,000 vehicles) and by the intersection geometry.

Trucks

The 50 percent probability of accepting a gap for left- and right-turning five-axle trucks at a high-volume intersection was generalized as 8.5 sec. The 10-sec gap was selected on the basis of the 85 percent probability of accepting a gap at a high-volume location for five-axle trucks, whereas the 15-sec gap was based on the 85 percent probability of accepting a gap at a low-volume intersection for five-axle trucks.

Field Study Findings Used in the Developed ISD Equations

The results of the field study findings substituted in the developed equations can be used to demonstrate the implications of the field study findings. The utility of the results is limited because field findings were based on only a few intersections. The equations developed on the basis of the GB84 ISD values allow for the inclusion of the observed speed reduction by the major-road vehicle. Equations in the GB90 do not include any terms for the deceleration of the major-road vehicle (even though the GB90 Figure IX-36 states that the sight distance allows for the major-road vehicle to reduce speed from design speed to 85 percent of design speed). The initial speed of the major-road vehicle was used in the design speed term in the equations.

Two sets of results were determined: one for passenger cars and the other for five-axle trucks (see Table 7). Sight distance values for right- and left-turn maneuvers were not separately determined because the calculated values would be very similar. Acceleration characteristics are similar for each turn ma-

TABLE 6 Sight Distances Based on Gap Acceptance Procedure

Speed (mi/h)	Passenger car sight distances (ft)			Truck sight distances (ft)		
	Gap accepted (sec)			Gap accepted (sec)		
	7.0	8.25	10.5	8.5	10.0	15.0
20	206	243	309	250	294	441
25	257	303	386	312	368	551
30	309	364	463	375	441	662
35	360	424	540	437	515	772
40	412	485	617	500	588	882
45	463	546	695	562	662	992
50	515	606	772	625	735	1,103
55	566	667	849	687	809	1,213
60	617	728	926	750	882	1,323
65	669	788	1,003	812	956	1,433
70	720	849	1,080	875	1,029	1,544

TABLE 7 Passenger Car and Five-Axle Truck Sight Distances Using Findings from the Field Studies

Speed V_s (mi/h)	Speed Reduction (mi/h)	Reduced Speed V_{rs} (mi/h)	Acceleration* Time t_t (sec)	Acceleration* Distance P (ft)	Deceleration Distance D_{dec} (ft)	Deceleration Time t_{dec} (sec)	Field Results FPC or FT (ft)	AASHTO B-2b&Cb or BT-2b&Cb (ft)	Percent Difference
Passenger Cars									
20	5	15	3.97	59	49	1.91	166	240	31
30	5	25	8.05	179	77	1.91	298	370	19
40	11	29	9.69	244	213	4.20	430	570	24
50	13	37	12.95	403	317	4.96	624	850	27
60	16	44	**	**				1,150	
70	20	50	**	**				1,550	
Five-Axle Trucks									
20	5	15	16.61	125	35	1.36	299	670	55
30	10	20	14.41	223	100	2.72	457	1,179	61
40	15	25	18.21	349	195	4.09	654	1,549	58
50	25	25	18.21	349	376	6.81	792	2,516	69
60	35	25	18.21	349	596	9.54	971	**	
70	40	30	22.01	502	801	10.90	1246	**	

* Acceleration time and distance values were determined from the following regression equations:

Passenger Car $P = 7.755\, t_t + 1.800\, t_t^2$ limits = 0 to 14.62 sec
Five-axle Truck $P = 1.524\, t_t + 0.967\, t_t^2$ limits = 0 to 23.37 sec

** Speed is beyond data limit.

neuver, and the increase in sight distance that the longer left, minimum turning path creates is less than 2 percent. Values selected for use in the calculations represent either the more conservative findings from the left- or right-turn maneuver or the findings that were based on a substantially greater amount of data.

The following study data were used to calculate intersection sight distances for turning five-axle trucks and passenger cars:

- Speed reduction values were based on observations made at two intersections with a high volume of five-axle truck traffic (one of which is a high-volume, urban intersection). The values used are given in Table 7.
- Vehicle acceleration time and distance values are from field observations of right-turning five-axle trucks and passenger cars at a high-volume, urban intersection. The values and regression equations are given in Table 7.
- The distance and time to decelerate values were determined using the 50 percent cumulative deceleration rate for major-road vehicles. Deceleration rates used for drivers reacting to a five-axle truck or passenger car are 3.67 and 2.62 mi/hr/sec, respectively. The 3.67 mi/hr/sec rate is within the range of comfortable rates listed in the GB84 and GB90.
- Minimum separation time between the rear bumper of the turning vehicle and the front bumper of the major-road vehicle was assumed to be 1.0 sec.

Other assumptions made for the analysis include the following:

- Perception-reaction time is 2.0 sec.
- Minimum turning vehicle radius is 40 ft for passenger cars and 60 ft for five-axle trucks.
- Length of vehicle is 19 ft for passenger cars and 55 ft for five-axle trucks.

Table 7 contains the analytic "field" results for passenger cars (FPC) and trucks (FT). Also given in Table 7 are the GB90 B-2b & Cb Curve intersection sight distance values as shown in GB90 Figure IX-40 and the results using the Green Book truck characteristics in the developed equations (BT-2b & Cb) (*7*). The passenger car results based on the field findings are between 19 and 31 percent less than the GB90 B-2b & Cb Curve. Truck results are between 55 and 69 percent less than the values from using the Green Book truck data in the developed equations (BT-2b & Cb values).

COMPARISON OF INTERSECTION SIGHT DISTANCE PROCEDURE RESULTS

Passenger Car Findings

The results from the gap acceptance procedure (G-7.0, G-8.25 and G-10.5) and the field studies (FPC) are shown in Figur 2. Also included in the figure are SSD from the GB90 an findings from field studies conducted by the Special Studie Unit of the Connecticut Department of Transportatio (ConnDOT) (*11*).

The field results (FPC) are between SSD and the 7.0-se gap results for speeds less than 40 mi/hr. Between 40 and 5 mi/hr, the field results are greater than both the SSD and th 7.0-sec gap results but are less than or near the 8.25-sec ga results. The 8.25-sec gap yields less than 800-ft sight distanc (for speeds less than 70 mi/hr), which is the value that Tunna

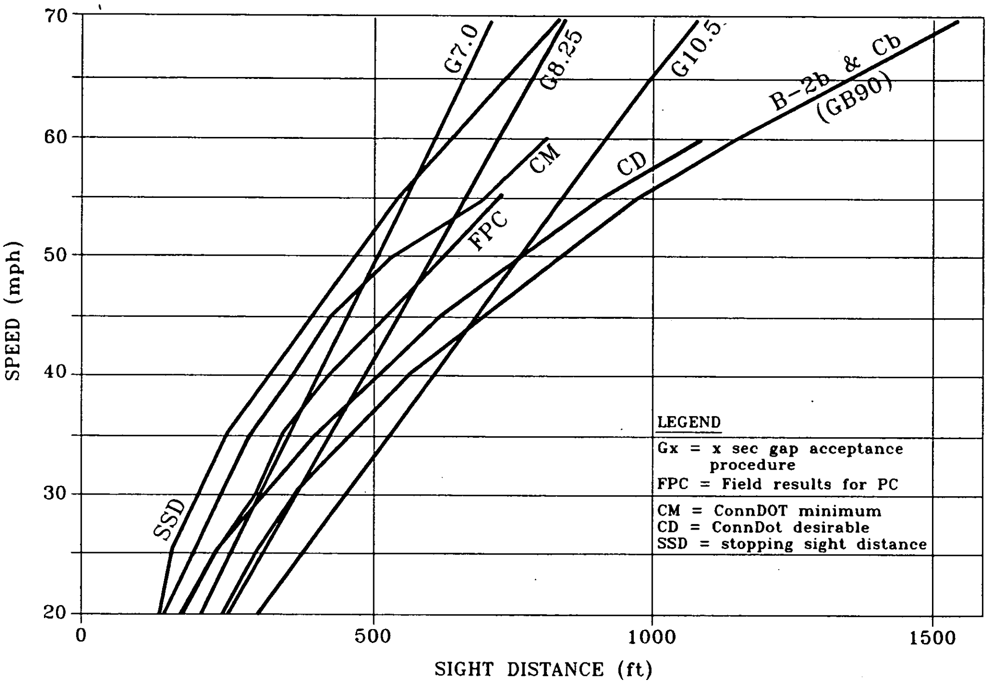

FIGURE 2 Comparison of passenger car sight distances.

and Pushkarev state is the limit of an individual's perception of vehicular movement (*10*).

ConnDOT's desirable ISD values are also based on field studies. In addition to the desirable values, minimum sight distance values based on stopping sight distance are cited. The 8.25-sec gap procedure produces sight distances that are higher than ConnDOT's minimum values for speeds less than 50 mi/hr and higher than ConnDOT's desirable values for speeds less than 35 mi/hr.

Truck Findings

Figure 3 shows the curves from the field results for trucks (FT) and the results from gaps studies (G-8.5, G-10.0, and G-15.0). The figure also has the GB90 passenger car SSD values (to represent the distance needed by a major-road passenger car to come to a stop on a wet pavement).

The field sight distance results are near the 10.0-sec gap results for speeds less than 55 mi/hr. They are also greater

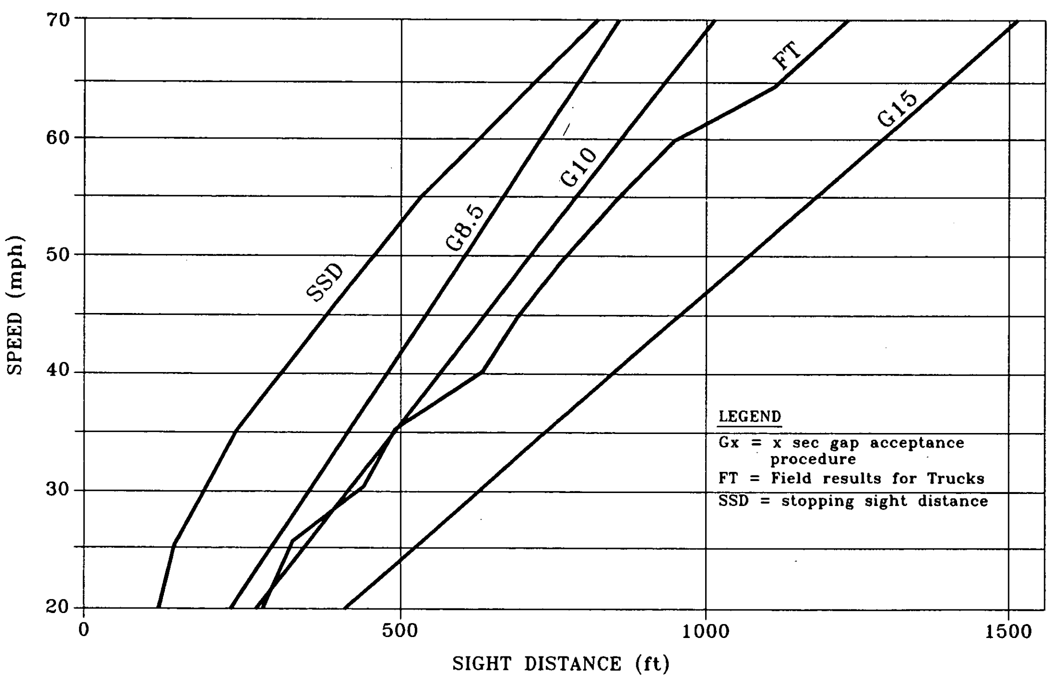

FIGURE 3 Comparison of five-axle truck sight distances.

than the SSD values. The 15.0-sec gap procedure produces significantly larger sight distance values than the results from the field observations. The gap procedure using a 10.0-sec gap produces sight distances less than 800 ft for major-road speeds less than 55 mi/hr.

SUMMARY

The Green Book states that after a vehicle has stopped at an intersection, the driver must have sufficient sight distance to make a safe departure through the intersection area. AASHTO sight distances for passenger cars and the sight distances for trucks (generated using the truck characteristics in the Green Book) produce values that are too conservative. These sight distances are approximately 35 percent greater than what passenger car drivers use and 63 percent greater than what truck drivers use as indicated by data from field studies.

An alternative approach may be to base the criteria on gap lengths safely accepted by the side-road vehicles. An initial field study at a few intersections indicates that representative sight distances are obtained using a critical gap of 8.25 sec for passenger cars and 10 sec for trucks. These gap values produce sight distances that are near the results developed from a field study in the developed ISD equations, which (with different input data) approximately reproduce the AASHTO ISD criteria. The 8.25- and 10.0-sec gaps represent the 85 percent probability of accepting a gap for both right- and left-turning vehicles at a high-volume, urbanized intersection.

CONCLUSION

Current sight distance procedures are a series of equations representing several interrelated maneuvers. To determine intersection sight distance, the user must make several assumptions on individual driver and vehicle performances (for example, perception and reaction time, acceleration, and deceleration) that are then combined to produce the sight distance value. An error or poor assumption for one parameter can have a major influence in the sight distances calculated. Several parameters are in need of frequent updating as the vehicle fleet changes. A gap acceptance procedure could simplify the process while implicitly considering the interrelated maneuvers being performed. Any future alternative procedures for determining intersection sight distances should also consider the driver's visual limitations.

Specifically, the following conclusions were determined from this research:

- Existing ISD procedures are difficult to reproduce, which can cause difficulties if input parameters need to be varied (for example, if a significant number of large trucks are expected at the intersection or if the cross street intersects a multilane highway).
- Input parameters could be in need of frequent updating to reflect the current vehicle fleet.
- A gap acceptance procedure for intersection sight distance for stop-controlled approaches would simplify the process.
- A comprehensive study should be conducted to determine the actual gap values on the basis of more data from sites with varying conditions, such as geometry, traffic characteristics, and volume levels, and from different driver groups such as inexperienced and older drivers.
- Further review is needed of driver visual limitations and factors (known and presently unknown) that could be used to select adequate ISD values before possible inclusion into future editions of the Green Book.

RECOMMENDATIONS FOR FUTURE RESEARCH

The field study methodology provided a practical and reasonable means for establishing estimates of the parameters used in the Green Book ISD procedures. Weaknesses lie in the limited data set and the loss of accuracy in visually detecting vehicles farther than 500 to 600 ft from the intersection. Nonetheless, the data needed to formulate ISD criteria can be established from actual field studies. A more comprehensive study would overcome the weaknesses in this study and provide additional support to modify the Green Book ISD procedures, select parameter values, and critically evaluate the feasibility of adopting intersection sight distance criteria based on gap acceptance.

Some of the critical gap values determined at several of the intersections were influenced by geometric or traffic characteristics. Additional research is necessary to measure the impact of different characteristics (e.g., rural versus urban location of the intersection, high versus low volume, grades on minor road, night versus daylight conditions) on the gap acceptance value. Additional research on the effects of different drivers, such as older drivers, on gap acceptance is needed, along with an evaluation of what percentile gap value should be selected (e.g., 50th percentile versus 85th percentile versus 99th percentile). Such findings could be incorporated into a descriptive gap acceptance model, which would provide design flexibility while ensuring that the attributes of functional classification are satisfied.

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REFERENCES

1. *A Policy on Geometric Design of Highways and Streets.* AASHTO, Washington, D.C., 1990.
2. *A Policy on Geometric Design of Highways and Streets.* AASHTO, Washington, D.C., 1984.
3. D. W. Harwood, J. M. Mason, Jr., W. D. Glauz, B. T. Kulakowski, and K. Fitzpatrick. *Truck Characteristics for Use in Highway Design and Operation, Volume I: Research Report and Volume II: Appendices.* FHWA-RD-89-226 and 227. Federal Highway Administration, Washington, D.C., 1990.
4. K. Fitzpatrick. *Sight Distance Procedures for Stop-Controlled Intersections.* Ph.D. thesis. Pennsylvania State University, University Park, 1989.

5. J. M. Mason, Jr., K. Fitzpatrick, and D. W. Harwood. Intersection Sight Distance Requirements for Large Trucks. In *Transportation Research Record 1208*, TRB, National Research Council, Washington, D.C., 1989, pp. 47–58.
6. J. M. Mason, Jr., K. Fitzpatrick, and D. W. Harwood. Field Observations of Truck Operational Characteristics Related to ISD. In *Transportation Research Record 1280*, TRB, National Research Council, Washington, D.C., 1990, pp. 163–172.
7. K. Fitzpatrick and J. M. Mason, Jr. Review of AASHTO Case III Intersection Sight Distance Procedures. In *Transportation Research Record 1280*, TRB, National Research Council, Washington, D.C., 1990, pp. 216–226.
8. K. Fitzpatrick and J. M. Mason, Jr. Review of AASHTO Green Book Sight Distance at Ramp Terminals Procedures. In *Transportation Research Record 1280*, TRB, National Research Council, Washington, D.C., 1990, pp. 190–198.
9. K. Fitzpatrick. Gaps Accepted at Stop-Controlled Intersections. In *Transportation Research Record 1303*, TRB, National Research Council, Washington, D.C., 1991, pp. 103–112.
10. C. Tunnard and B. Pushkarev. *Man-Made America: Chaos or Control?* Yale University Press, 1963.
11. *Parameters Affecting Intersection Sight Distance*. Internal Report. Connecticut Department of Transportation, Special Studies Unit, Oct. 1985.

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Safe Accommodation of Pedestrians at Intersections

MARTIN T. PIETRUCHA AND KENNETH S. OPIELA

For years there has been an independent approach to the design of highways and streets to accommodate vehicle and pedestrian movements. The highway and street design process in the AASHTO geometric design policy is examined from a pedestrian design perspective to determine the adequacy of highway design standards in considering the pedestrian, the appropriateness of current design treatments, the compatibility of pedestrian facility designs and highway facility designs, and the effectiveness of the various treatments. Where applicable research related to pedestrian operations and safety might be incorporated into intersection design is discussed. Although the Green Book is a policy and comprehensive coverage of all topic areas is not possible, some changes and short additions in areas such as sidewalks/walkways, refuge islands, and sidewalk flares are suggested to improve the information available to the designer.

Allowing vehicles and pedestrians to share the roadway environment safely and efficiently is not an easy task. The characteristics of these modes of travel are vastly different, and yet they compete for use of the same street and highway space. Typically there have been independent approaches to the design of highway and pedestrian facilities. In many places, highway congestion and pedestrian safety problems have become prevalent, indicating a critical need to assess the design process and search for effective means to integrate these independent design efforts. The American Association of State Highway and Transportation Officials' (AASHTO's) *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 decisions in the United States. Therefore, it is appropriate to examine this document to determine the potential for a broader view of the highway design process such that the needs of pedestrians are adequately addressed.

This paper examines the highway and street design process in the context of pedestrian needs. It attempts to determine

- Whether highway design standards adequately consider the pedestrian,
- Whether current design treatments are appropriate,
- Whether pedestrian facility designs are compatible with highway facility designs, and
- Needs for future research.

This paper focuses on the design of intersections where the competition for space is the most critical.

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BACKGROUND

Effective roadway design involves establishing realistic design criteria and controls for the traffic being served. The designer must understand the full range of traffic to be accommodated by a design. For motor vehicles, this involves knowing the number and types of vehicles that will use an intersection. In the case of the pedestrian, this implies an understanding of the number and characteristics of the pedestrians who will use an intersection.

The AASHTO Green Book is one of the most widely used reference manuals on the design of highway facilities. The Green Book was published in 1984 as an update and consolidation of previous documents on the subject, and the 1984 edition was updated in 1990. The Green Book provides highway design guidance for the full realm of highway types and situations. Whereas it recognizes the needs of the pedestrian, the current edition does not adequately address all of the aspects related to providing safe and convenient conditions for the pedestrian. Since the Green Book is intended to be a policy, it cannot be expected to address every detail important to pedestrian-related design. However, more reference to pedestrian needs is essential to raise the consciousness of highway designers to pedestrian needs. The following sections provide some background on the general guidance provided by the Green Book relative to pedestrians.

CRITIQUE OF CURRENT GENERAL DESIGN GUIDELINES RELATED TO PEDESTRIANS

Highway Functions

Functional classification is used in the Green Book as the basis for determining the applicable criteria for street and highway design. The functional classes are defined in broad terms of movement hierarchies and relationships. In making these definitions, however, there is no mention of the movement of pedestrians. Whereas there may not be an exact pedestrian parallel for the AASHTO-defined six stages of a trip (i.e., main movement, transition, distribution, collection, access, and termination), elements of a pedestrian trip or a trip in which pedestrian movement represents an important element (e.g., the distribution or collection elements of a transit trip) certainly fit into this hierarchy. In the Green Book, the movement hierarchy sets the framework for the functional relationships for the vehicle trips. There may be a hierarchy associated with pedestrian trips, but this may be less critical than the presence or absence of pedestrians relative to design criteria for a roadway within each of the functional classes. This suggests that within each functional class there should be subclasses that reflect the impact of the presence of pedestrians on highway design and operation. Such a classification scheme would permit more rational guidance for the design of roads for the movement of both vehicles and pedestrians. In the absence of such criteria, the Green Book treats pedestrians in a haphazard manner. For example, whereas several of the chapters give limited guidance on provision of pedestrian access and the desirability of sidewalks, no specific criteria are offered to provide a basis for design decisions or to provide the justification needed for more extensive pedestrian treatments in special situations (e.g., around schools and near shopping areas).

The nature of pedestrian trips, and concomitantly some of the design decisions, vary by situation and somewhat according to the functional classification of the road. For example, lunchtime shoppers along an urban minor arterial have much different user characteristics and trip purposes than do schoolchildren walking along a local collector street to get to their school bus stop or commuters walking along the collector to get to the transit stop located on an arterial street. Highway designers need to recognize different pedestrian uses to make the most efficient use of the right-of-way to serve all uses. Beyond the need to serve different users, the designer must apply design criteria that promote safety and convenience. For example, the need for sidewalks and wide shoulders varies between collector and arterial streets to reflect differences in access. Similarly, the frequency of pedestrian crossings may dictate that medians become the norm for the urban arterial.

The need exists to establish guidelines for highway design that would apply where pedestrians represent a significant proportion of the traffic. For example, design and operational strategies for such a situation might be to control vehicular speeds, minimize vehicular impedance to the pedestrian, minimize pedestrian-vehicle conflicts, reduce conflicting attention demands, ensure adequate walkway separation, and provide aesthetic designs. These desired pedestrian pathway attributes would be emphasized rather than movement of vehicular traffic. Experience has shown that designers can find ways for vehicles and pedestrians to safely and conveniently coexist (*2,3*).

Pedestrian Characteristics

The Green Book recognizes the influence of pedestrian physical and behavioral characteristics on street and highway operations. Chapter 2 deals with the general characteristics of traffic and describes body area, walking rates, and walking capacities of pedestrians. Information is provided on average pedestrians as well as those with physical, visual, or mental handicaps. Walking capacities are discussed for sidewalks, stairways, and intersections, and two simple models are provided for determination of required pedestrian storage area at intersections, crosswalk widths, and level of service.

The Green Book accurately states that pedestrians are a major consideration of every roadway environment, urban and rural. However, it goes on to emphasize that they will most likely be found in urban areas, and, therefore, it is the urban pedestrian that most often influences design. There is a measure of truth in this statement, but walking occurs in suburban and rural areas, sometimes in significant volumes (*2,3*). However, the distinction between urban, suburban, and rural areas is not always clear. A consequence is that pedestrian movements take place in areas where they are not expected by drivers, and the roadway facilities do not adequately accommodate these movements. For example, in areas being developed on the suburban fringe, the pedestrian population may be especially at risk because sidewalks or shoulders have not been provided and traffic operates at higher speeds. Even where development is intense, an automobile orientation results in wide arterial cross sections, limited facilities for crossings, and barriers that prevent convenient pedestrian movement.

The Green Book may also indirectly discourage consideration of pedestrian needs. It describes the shopper as a pedestrian who is influenced chiefly by the "weather or advertised sales." They are described as "unpredictable, obstinate, ignorant, inattentive, or defiant." Some of this may be true, but one must also consider that pedestrians are unprotected, slow moving, and extremely fragile.

Physical Characteristics

The human body dimensions (an ellipse of 24 × 18 in.) in the Green Book are based on the work of Fruin (*4*) and are consistent with those used in the 1985 *Highway Capacity Manual* (HCM) (*5*). There is widespread agreement on these dimensions, but they do not take into consideration the increased body ellipse needs of the elderly with canes or walkers or adults with shopping carts or baby carriages. The need to design for this element of the pedestrian constituency may be small, but the designer should be aware of these diverse user groups in some situations.

The Green Book states that walking rates are generally 2.5 to 6.0 ft/sec with an average of 4.0 ft/sec in accordance with the 1988 *Manual on Uniform Traffic Control Devices for Streets and Highways* (MUTCD) (*6*). It even goes so far as to state that in areas where there are many older people, a walking rate of 3.0 ft/sec should be considered. However, other studies have shown a wider range in walking rates. McShane and Roess suggest a range of approximately 2.5 to 8.0 ft/sec (*7*), and Bowman et al. give an average walking rate of 4.5 ft/sec (*8*). In fact, if one assumes a walking speed of 4.5 ft/sec to determine pedestrian clearance interval, 78 percent of the pedestrians observed in one study would have to quicken their walking speed to safely cross the street during the green indication. A current Federal Highway Administration (FHWA) study dealing with older-pedestrian characteristics should yield some useful information about this subject (Contract DTFH61-91-C-00028, being performed by the Center for Applied Research, Inc., Great Falls, Virginia).

Handicapped Pedestrian Characteristics

It is correctly noted in the Green Book that pedestrians with ambulatory difficulties are especially sensitive to stairs, curbs, or other horizontal obstructions that are in their paths. Recent research shows that such pedestrians are also sensitive to the type and condition of the walking surface. Kulakowski et al.

(*9*) have found that handicapped walkers require higher levels of walking-surface friction than the nonimpaired walker. The more important finding may be that the friction needed to safely traverse these pathways frequently exceeds the obtainable friction coefficients for many walking surfaces. These findings may also have implications for the section of the Green Book dealing with pavement surface types.

As stated in the Green Book, pedestrians with vision problems require special consideration, but these are not limited to just the texturized pavement marking treatments mentioned therein. There is some indication (based on a relatively small sample of blind pedestrians) that visually handicapped pedestrians prefer curb corners to be more angular (i.e., shorter comer curb radii) to give them better directional orientation around the intersection (*10*).

Walkways

The Green Book outlines the design requirements for walkways. It notes that walkways or sidewalks are commonly considered as part of the street cross section in urban areas, but few are provided in rural areas. The Green Book suggests that the need in rural areas in great because of higher speeds on rural roads and the general inadequacy of lighting. It recommends that sidewalks or walkways be installed in rural and suburban areas to connect community activity areas (i.e., schools, shopping, and residential areas). It notes that sidewalks may not be required in the initial development stages but that adequate provisions for future inclusion are needed. Shoulder treatments may suffice temporarily, but the right-of-way planning should include space for a sidewalk separated from the shoulder, a buffer or border area, and adequate clearance for adjacent land uses. Sidewalks are recommended to be 4 to 8 ft in width with a minimum of 2 ft of clearance or separation. The justification for providing such pedestrian facilities in suburban and rural areas should be based on the volumes of traffic, the relative timing of demand, and speeds.

The Green Book provides other recommendations for general practice relative to walkways, including the following:

- Sidewalks should be provided along any highway without shoulders to keep pedestrians from using the traffic lanes.
- Crosswalks should be marked where the pedestrian paths between community activity areas intersect streets and highways.
- The connectivity of walkways should be maintained where community activity areas are close together.
- Walkway designs should accommodate the elderly and handicapped.
- Special treatments are needed for bridges to provide a safe transition and to provide railings and fences along the bridge walkpath.
- Barrier curbs should be provided on low-speed roads and full barriers on high-speed roads where the pedestrians are close to the traffic.

These guidelines, while useful, provide neither adequate quantitative criteria nor full integration into the facility design process.

The Green Book offers information related to the capacity of walkways but does not address the more significant pedestrian capacity problems at intersections. As with the roadway, the system bottlenecks usually occur at the node points in the network.

Although the Green Book gives a short description of pedestrian flows and operations at an intersection, it does not give the designer an adequate feel for what can be expected in these situations. The Green Book briefly discusses holding areas for queued pedestrians, crosswalk widths, and the use of traffic control devices to create gaps in the traffic stream that will allow for pedestrian crossings. This would lead the designer to think that pedestrian capacity is based solely on area or space considerations. The capacity analysis technique prescribed by the HCM (*5*) is based on a time-space concept that considers pedestrians waiting to cross the street, pedestrians crossing the street, and pedestrians who use the corner to circulate through the pedestrian network without entering the street (i.e., people walking around the corner). Each of these categories places a different demand on the time-space capacities of the corner (e.g., pedestrians waiting to cross the street take up more time and less space than pedestrians moving around the corner who need more space but less time). The current policy should be revised to give a fuller description of the technique so that the designer is aware that pedestrian volumes and pedestrian flow rates are important factors in the design process.

Crossings

The Green Book provides some guidance relative to pedestrian crossings, but this information deals mostly with separated crossings. It suggests that separated crossings may be required where pedestrian volume, traffic volume, intersection capacity, or other conditions "favor" such a treatment. It suggests that an individual study of each situation is needed to consider heavy peak movements, abnormal hazards, or a combination of these factors to decide on the need for a separated crossing. Several general design guidelines for over- and underpasses for pedestrians are provided.

CRITIQUE OF CURRENT DESIGN GUIDELINES FOR AT-GRADE INTERSECTIONS

After establishing the basics of functional classifications and design controls in the first two chapters, the Green Book addresses elements of design, cross-sectional features, and specific design standards for local roads and streets, collector roads and streets, rural and urban arterials, and freeways. There are valid pedestrian concerns related to each of these areas that would warrant a careful scrutiny of these chapters, but this paper focuses on Chapter IX, At-Grade Intersections.

Intersection Types

The discussions of the various types of intersections in this section make only limited mention of pedestrians. A simple discussion of how pedestrians would function as part of these different intersections would go a long way toward sensitizing the designer to pedestrian needs. For example, there is a discussion of how skew affects the operation of the intersections from a turning vehicle perspective; however, there is no mention of how skew will affect pedestrian operations (e.g., create longer crossing distances and greater pedestrian exposure).

The Green Book describes various types of basic and enhanced intersection designs, but it does not consider the variety of design treatments that have been used to accommodate pedestrians in real-world situations. For example, sidewalk flares (see Figure 1) can be constructed. They offer the advantages of shorter crossing distances, more pedestrian visibility, a better view of traffic for pedestrians, and sidewalk space for queuing, but reduced right-turn capacity and corner drainage problems are disadvantages. Coverage of design considerations for this type of treatment is given by Pietrucha and Plummer (*11*).

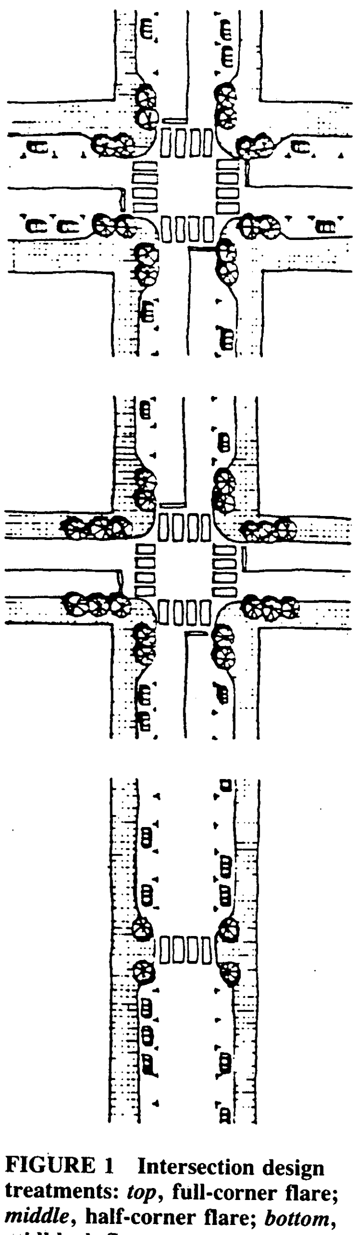

FIGURE 1 Intersection design treatments: *top*, full-corner flare; *middle*, half-corner flare; *bottom*, midblock flare.

Capacity Considerations

The Green Book indicates that the vehicular capacity of intersection designs needs to be evaluated using procedures outlined in the HCM (*5*). The HCM also offers an extensive treatment of pedestrian capacity for sidewalks, crosswalks, and street comers. Procedures are presented in the HCM for applying the level of service (LOS) criteria outlined to walkways, queuing areas, street corners, and crosswalks. The latter two are noted as the most difficult to analyze due to the diversity of movements and the high volume of pedestrians. The procedures outlined allow for the determination of a LOS measure based on pedestrian volumes, walkway width, crossing times, crosswalk areas, surge factors, and circulation times. Since the design process typically involves iterative analyses of capacity adequacy and design feasibility, the need exists to develop an integrated approach that will include both pedestrian and vehicular capacity considerations.

Alignment and Profile

The Green Book initiates the section on alignment and profile by noting that intersections are points of conflicts between vehicular and pedestrian traffic juxtapositioned on a specific environment. It is certainly true that vehicle-pedestrian conflicts do occur at intersections; however, no further guidance is provided to ensure that intersection alignments are set in a manner that enhances the safety and convenience of the pedestrian. For example, the Green Book recommends the intersection of roadways at 90-degree angles. However, it should also mention that this design standard represents the best option for both pedestrian and vehicular traffic. Sight lines are optimal, conflict space is limited, and crossing distances (and hence exposure time) are reduced.

The profile of intersection approaches receives only cursory attention, but it is another complicating factor. The sight lines of traffic approaching an intersection on a significant grade are compromised. This limits the opportunities for the pedestrian and the motorist to assess a situation.

A fundamental problem with the Green Book is that its criteria are specified in a two-dimensional format. Whereas this may be a necessity for presentation, in reality the sight lines are further complicated by other objects at or near the intersection and the viewing position of the driver or pedestrian. For example, the view of a driveway or sidewalk for a truck driver with an eye height of 6 ft may be impaired by trees planted along the right-of-way. A two-dimensional view might otherwise indicate adequate sight lines. Pedestrians in a crosswalk may not be fully visible in such a situation.

Intersection Curves and Turning Radii

Chapter III of the Green Book provides guidance on the design of curves. Fundamentally, an appropriate curve radius is a function of traffic speed and volume and traffic mix. The Green Book presents appropriate turning radii for various vehicle types. The critical factor is inner wheelpath during a turn, particularly for long vehicles. The Green Book specifies an envelope for turns by larger vehicles, but it does not define a buffer zone that would help define an adequate intersection design. This buffer zone would indicate an area unsafe for pedestrians. It should be something greater than the inner wheelpath, since not all drivers negotiate a curve in the same manner. The Green Book indicates the potential need to acquire additional right-of-way to accommodate pedestrians at intersections and preferred sidewalk-curb radii at intersections. These are discussed only in the context of urban arterial intersections.

The Green Book also sets standards for the design of intersections with turning roadway elements. These can pose a hazard to pedestrians, since they promote faster traffic speeds. The Green Book does not offer any guidance on the optimal location of pedestrian crossing on these, although it is generally accepted that right angle crossings are the best.

Islands and Medians

Medians and refuge islands (see Figure 2) are other features important to pedestrians that have not been fully described in the design standards. They reduce crossing delay and increase pedestrian safety by separating conflicts, controlling angle of conflict, reducing pavement areas, regulating traffic

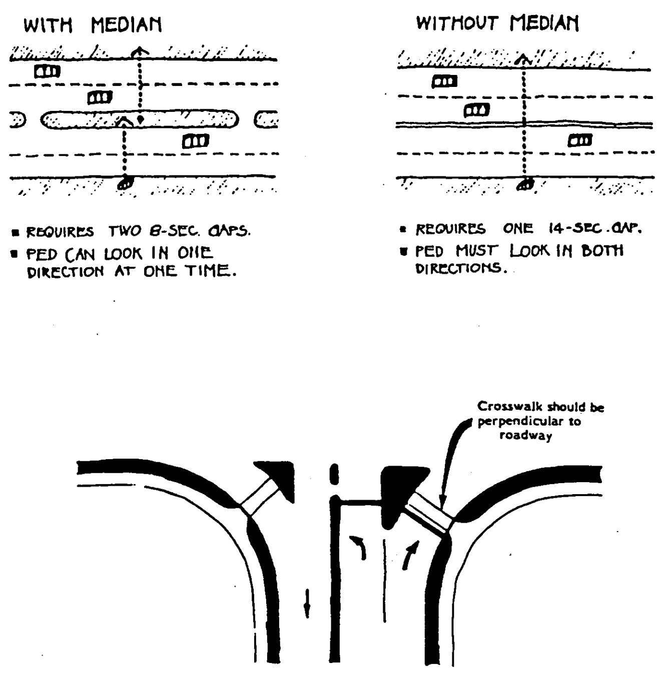

FIGURE 2 Typical intersection design treatments to accommodate pedestrians.

by indicating proper use of an intersection, favoring particular turning movements, protecting pedestrians, protecting and storing turning and crossing vehicles, and providing space for the location of traffic control devices.

The Green Book describes various types of intersection islands and offers guidance on their use but provides only limited attention to refuge islands. Its principal deficiency is the lack of design details and criteria for the use of refuge islands. The size, configuration, and integration of other elements need to be addressed in the context of the number and types of pedestrians that will be using the facility and the type of road on which it is placed. This section also provides inadequate examples of the proper use of refuge islands (e.g., Figure 2).

Generally, pedestrian refuge islands should be used to facilitate the movements of pedestrians across wide streets or to protect pedestrians in areas of the intersection where there may be complicated (e.g., irregularly shaped, skewed), confusing traffic flow patterns, or segregated, high-volume vehicle movements (e.g., turn lanes). The island also provides a stopping point for the slow walker who cannot cross the entire street in the allocated pedestrian time. At isolated signal locations, the reduced pedestrian clearance time can minimize the signal cycle length and the overall delay to vehicular traffic. It has been proposed that pedestrian refuge islands be provided when the total length of a crosswalk is greater than 75 ft or in areas where there are many elderly or handicapped pedestrians. Refuge islands should be provided if the intersection cannot be crossed in the walk/green time allotted for the pedestrian movement using an assumed walking rate of 3.5 ft/sec (*8*). This also assumes that the signal timing cannot be changed to accommodate these special pedestrians. Islands should be designed to make the approach clearly visible, allow sufficient time for driver and pedestrian decision making, and ensure that the path and approach conditions follow the natural path of movement.

The Green Book offers information regarding minimum sizes for refuge islands based on the need to give pedestrians a sense of security when they are near moving traffic. The recommended minimum is 4 ft and the preferred width is 6 ft for pedestrian refuge islands (*8*). The length of the refuge island should be a function of the use of the island; however, median-type refuge islands should be at least 12 ft long (*8*).

If the refuge island is to be a raised barrier curb design, there must be a pedestrian travel path through the island, or there must be sufficient space for curb ramping and a level waiting area large enough for a wheelchair (*8*). The objective in the design of islands from a pedestrian point of view is to provide a traversable path. The designer needs to be mindful of details such as curb heights (for older and younger pedestrians); pedestrian volumes; surface types; and locations of traffic signal hardware, luminaire supports, ramps, and inlet grates.

The Green Book chapter Cross Section Elements describes a median as a device that controls vehicle movements or stores stopped vehicles. This discussion should include information on the use of medians as refuge areas for pedestrians. One of the major problems that pedestrians face, especially older pedestrians, is the crossing of wide streets. At unsignalized intersections, medians allow the pedestrian to perform a simplified crossing task. The pedestrian can accept smaller gap in the traffic stream, and the task is simplified by having to locate gaps in traffic coming from only one direction (Figure 2). At signalized intersections, because of traffic control considerations, the available pedestrian crossing time is usually the minimum specified in the MUTCD and the *Traffic Control Device Handbook* (*12*). Many pedestrians do not walk at speeds used for design purposes (4.0 ft/sec), and slower walkers have very little time to traverse a wide intersection. If the minimum pedestrian indication times were set at lower walking speeds, the time allocated for this "minor" traffic movement would put the major traffic movement at a disadvantage. In cases where it is desirable to maintain a minimum pedestrian crossing indication, a median would provide a place for pedestrians to safely stop and continue their crossing during the next pedestrian indication. Whereas this type of operation is less than ideal, especially from the pedestrians' point of view, it provides a solution to a common problem. The need also exists for treatments to discourage improper use of medians by pedestrians (e.g., pedestrians can be observed walking laterally along medians to capture the next available gap in traffic, which violates a basic tenet of traffic engineering—control the number of possible conflict points).

Traffic Control Devices

The Green Book appropriately indicates that design and control treatments for intersections must be developed in concert. The geometry and cross section of an intersection need to be considered in conjunction with options for traffic control devices (TCDs)—signals, signs, pavement markings, and street furniture. The nature of the vehicular and pedestrian traffic must, of course, dictate the selection of appropriate design and control treatments. The following paragraphs discuss some of the interactions associated with pedestrian-sensitive design and control of intersections.

Flow Control

It is well known that TCDs can be used to create artificial gaps in the vehicular traffic so that pedestrians can cross the road. The policy states that if there are no controls the pedestrian must wait for a sufficient gap to cross. However, there is no guidance for the designer regarding the definition of a sufficient gap. Analytically, one could reason that a sufficient gap could be based on the pedestrian crossing distance, d_c (ft), and an assumed pedestrian walking speed, s_p (ft/sec). This would yield a gap time that is equal to d_c/s_p (sec). There is also a need to include some amount of pedestrian start-up time (t_{psu}, sec) in this measure of gap time [e.g., 4 to 7 sec (*6*)]. As with intersection sight distance calculations, this time could then be converted into a pedestrian sight distance requirement (d_p) based on roadway design speed. This concept could have serious implications on how highway designers determine intersection sight distance.

For example, consider the intersection sight distance requirements at a stop-controlled intersection for Case IIIB, Turning Left into a Major Highway, in the Green Book (*1*). This condition is assumed because it gives the maximum intersection sight distance dimension that can be required at an intersection. If one assumes a roadway with a 50-ft curb-to-curb width and a design speed of 50 mph and determines from the appropriate sources in the Green Book the values for t_p and P, the required intersection distance to the right of the turning vehicle would be approximately 725 ft:

$$d = Q - h \tag{1}$$

where

d = required passenger car sight distance to the right along the major road from the intersection (ft);
$Q = 1.47(0.85)V\ (J + t_p)$ (for this example the value is calculated to be 1,062 ft);
$h = P - 16 - [1.47 * 0.85 * V\ (J)] - L$ (for this example the value is calculated to be 340 ft);
V = design speed of the major highway (mph) (for this example, the value is 50 mph);
J = sum of the perception time and the time required to actuate the clutch or an automatic shift [for this example, the value is assumed to be 2 sec (*1*)];
t_p = time required to travel the distance P (sec), taken from Green Book Table IX-7 (*1*) (for this example, the value is estimated as 15 sec);
P = distance for the turning vehicle to reach a speed 85 percent of the mainline design speed, taken from Green Book Figure IX-34 (*1*) (for this example, the value is 500 ft); and
L = overall length of the vehicle (ft), assumed to be 19 ft for passenger cars (*1*).

If one follows the conceptual argument for pedestrian sight distance, which is advanced above, for the same set of conditions the required pedestrian sight distance would be approximately 1,215 ft:

$$d_p = 1.47\ V[(d_c \div s_p) + t_{psu}] \tag{2}$$

where

d_p = required pedestrian sight distance to the right along the major road from the intersection (ft),
d_c = pedestrian crossing distance (for this example, the value is 50 ft),
s_p = pedestrian walking speed [for this example, the value is assumed to be 4 ft/sec (*1*)],
V = design speed of the major highway (mph) (for this example, the value is 50 mph), and
t_{psu} = pedestrian start-up time [for this example, the value is assumed to be 4 sec (*6*)].

As shown in this example, the required intersection sight distance for the vehicles turning left is considerably less than the required pedestrian intersection sight distance.

Even if the pedestrian could make the crossing (under the same lane width and design speed conditions) in two stages (breaking up the crossing into two parts by means of a pedestrian refuge island), the pedestrian sight distance to the left would still be less than the maximum required vehicle sight distance [approximately 735 ft for the pedestrian maneuver compared with approximately 725 ft for a vehicle turning right onto a major road from a standing start (Case IIIC, Turning Right into a Major Highway, in the Green Book (*1*)]. One may argue that the difference is hardly worth noting. However, one must keep in mind that the values for walking speed and pedestrian start-up time used in this example are in the conservative range. This whole argument might be considered moot when one considers a pedestrian's ability to see and detect motion of an object that is more than 700 ft away.

Device Application

Chapter IX of the Green Book is not intended to give exhaustive coverage to TCDs, and it appropriately reminds the designer that space must be provided for TCDs, including those for the pedestrian. Two important features that could influence the design of the intersection should be covered in this Green Book chapter. A properly designed intersection cannot function without properly designed pavement markings, in particular, crosswalk markings and stop bars. The size and placement of these devices could influence the size or placement of other geometric features of the intersection. They should also be considered as possible future additions to any intersection.

Although specific design requirements are not given for other TCDs, some general guidance concerning the size and placement of crosswalks and stop bars should be provided for the designer. Generally, crosswalks should be at least 6 and preferably 10 ft in width. There cannot be much choice about the length; however, for long and hazardous crossings, the crosswalk may need to be augmented with other devices such as curb flares or refuge islands. Stop bars are used to stop vehicles from encroaching on the crosswalk. They are usually 12 to 24 in. wide. They are placed 4 ft in advance of the crosswalk. In many instances, a staggered setup can be useful in providing better sight lines for vehicles that are turning right on red.

As with many of the other sections of the Green Book, the discussion on traffic barriers focuses entirely on vehicular traffic. Many different types of barriers provide an effective means of channeling pedestrian flows and prohibiting, or at least making more difficult, undesired pedestrian movements. Zegeer (*10*) presents a series of conditions in which barriers may be most useful along with a listing of potential advantages and disadvantages. Whereas this information should not be a principal focus of this section, it certainly deserves some mention in the Green Book.

Other Controls

Some analyses have shown that 50 percent of all urban pedestrian accidents involve dashes into the street at midblock locations or intersections (*11*). A frequently cited contributing factor with these types of accidents is that the motorists or pedestrians could not see each other because of on-street, parked cars. The designer should consider the prohibition of on-street parking near intersections. In cases where vehicle travel speeds are 35 mph or greater, it has been recommended that this distance be at least 100 ft (*8*). This section of the Green Book needs to be revised to include some discussion of pedestrian accident problems associated with on-street parking.

Similarly, turn restrictions can be considered to control traffic at an intersection. The provision of indirect left turns and indirect U-turns creates a wider crossing area than standard, small-radii turns. There should be some mention of the problems these types of designs can create for pedestrians in the Green Book.

The Green Book does not address traffic controls for school zones. Only general guidelines are provided for the use of flashing beacons, crosswalks, traffic signals, and grade separation structures. Some coverage of this special pedestrian group analogous to what is covered in the MUTCD should be considered.

Channelization

The Green Book offers a short section on channelization in Chapter IX. It makes reference to considering pedestrians, but the 10 principles presented for channelization fail to mention the pedestrian. A similar set of points would be appropriate to reflect the pedestrian's perspective of the intersection.

Cross Section Design

The entire section of the Green Book dealing with cross section elements considers design in terms of vehicular traffic only. Whereas many suburban and rural roads have little pedestrian traffic, people do walk on these roads. This is evidenced by the fact that approximately 15 percent of the pedestrian accidents in these areas occur when a pedestrian is struck while walking along a roadway (*2*). Design criteria related to several cross section elements should be reexamined to see whether changes need to be made to accommodate the pedestrian.

Pavement Cross Slope

The Green Book specifies that the normal pavement cross slope can range from 1.5 to 6 percent for properly draining the pavement. The higher values in this range may make walking a path parallel to the centerline of the roadway awkward, causing the pedestrian to walk where the slope flattens out—that is, nearer to the traveled way.

Lane Widths

The Green Book principally addresses the subject of lane widths in terms of the vehicular traffic because of their effect on highway capacity. Better guidance regarding the widths of lanes, especially the use of narrower lanes (i.e., 10 or 11 ft), is needed since the roadway is often shared by pedestrians and vehicles.

Shoulders

In the Green Book, the shoulder is only thought of as an area for stopped or disabled vehicles and as a structural element that laterally supports the roadway. Even though the use of the shoulder by pedestrians is to be discouraged in many situations, it must be realized that, despite the misgivings of the highway designer or traffic engineer, people walk there. Any design decisions regarding shoulders should consider pedestrian use and be made in concert with decisions regarding lane widths.

Curbs

This section of the Green Book should remind the designer that a curb is also a barrier to some pedestrians. Handicapped and older pedestrians find it difficult to negotiate high sections of barrier curb. Whereas pedestrian ramps can be part of a solution to this type of problem, they are not referenced in this section of the Green Book.

Walkways

Whereas the Green Book correctly states that there are no pedestrian or vehicular traffic based sidewalk warrants, a set of guidelines for the installation of sidewalks has been proposed by Knoblauch et al. (*13*). These guidelines are based on a study of pedestrian accidents related to pedestrian exposure and certain operational and design features. The study makes recommendations relative to land use category, roadway functional class, and development density for both new and existing urban and suburban streets. Because there are no formal guidelines for providing sidewalks in rural areas, the direction given in the Green Book provides sound but not definitive advice.

Wheelchair Ramps at Intersections

In the Green Book, the designer is referred to Chapter IV for direction in the design of wheelchair ramps at intersections. The treatment of this topic is woefully inadequate. Simple, effective designs are plentiful; however, inconvenient, poor-draining, dangerous ramps abound. For more complete coverage of this important topic, the designer should be referred to the FHWA implementation manual on this subject (*14*).

Driveways and Access Management

Chapter IX of the Green Book includes a section on driveways and notes that this type of feature is, in effect, an intersection. A serious deficiency of the Green Book is its treatment of driveways. The Green Book provides only limited attention to measures that can be applied to control access along major roadways. The goal of access control is to provide improved traffic flow and increased safety on streets and highways. Whereas sidewalks may have a consistent surface material across driveways, there is little else to warn, direct, or control the flow of vehicular or pedestrian traffic. Access control measures should be considered in the design of highways for the benefit of both vehicular and pedestrian traffic.

SUMMARY

This paper critiqued the AASHTO Green Book, focusing primarily on Chapter IX, At Grade Intersections. The fundamental findings of this critique are as follows:

1. The Green Book, though not silent on the subject of pedestrians, provides only limited guidance to the highway designer. It describes basic characteristics but does not cover the range of characteristics or the extent of pedestrian travel.
2. The Green Book is predominantly vehicle oriented. In many cases, provisions for pedestrians or other modes that share the highway right-of-way appear to have been added as afterthoughts.
3. The Green Book does not present a fully integrated approach to highway design. The nonmotorized modes, in particular, are not given adequate treatment, nor are the safety, traffic flow, or cost aspects of these modes given sufficient attention.
4. The Green Book is developed around a functional classification scheme related to vehicular traffic. This scheme does not effectively incorporate considerations for nonmotorized modes or the transitional nature of the highway environment.
5. The Green Book needs updating to reflect recent design and warrant criteria related to pedestrian facilities.
6. The Green Book provides only limited coverage of design treatments for intersections, islands, and medians. More attention is needed to promote their application to address pedestrian needs.
7. Separate sections dealing with pedestrians are not necessary. Passages regarding the pedestrian's place in the movement hierarchy and the functional relationships could be woven into the existing text.

It is not expected that highway designers will become experts about pedestrian design, but it is recommended that they devote more attention to the pedestrian as part of the total design process. There are similarities and differences in the design considerations for highways and pedestrians, as noted in Table 1. For example, the principles that apply to highway channelization apply to pedestrians as well. The vulnerability of the pedestrian in the intersection environment dictates that the highway design process, at least, explicitly assess the pedestrian. Ideally, the intersection design process would use a dual perspective approach to channelization that would better integrate all modes. These differences in design consideration are critical to the safety and convenience of the pedestrian. For example, crossing a major arterial poses a major risk and difficulty for pedestrians. Some approaches to the design of arterials focus on maximizing the provision of traffic lanes within the right-of-way. The provision of medians limits the lane configuration options and complicates the design, but it offers clear advantages to the pedestrian. Improvements to the Green Book that would encourage or help the highway

TABLE 1 Facility Design Criteria

	Considerations Associated with Design Elements	
	Roadway	Pedestrian Facility
ROW Width	o Capacity dictates number of lanes o Standard widths by functional class o Minimize land acquisition costs o Future improvement options	o Fruin's work defines required space o Amenity space desirable o Pathway alignment influenced o Pathway separation influenced
Lane Configuration Cross Section Number of Lanes Medians	o ROW limits o Capacity dictates number of lanes	o Curbs an impediment to pedestrians o Barriers may enhance pedestrian safety
Horizontal Alignment	o Terrain usually dictates o Adequate sight lines	o Minimize grade differentials o Locate crossing to maximize visibility
Channelization	o Minimize conflicting movements o Enhance capacity o Minimize delay o Enhance signal effectiveness	o Islands provide refuge to pedestrians o Pedestrians must deal with faster traffic o Queuing space affected
Drainage	o Minimize surface water	o Minimize puddles o Avoid placing drainage structures in pedestrian paths
Vertical Alignment	o Provide safe stopping distances o Provide adequate drainage o Provide smooth transitions between grades	o Pedestrian exposure may be more critical o Locate crossings with adequate sight lines
Turning Radii	o Increased radii mean higher speeds o Facilitates the turning of larger vehicles	o Increases pedestrian walking distances o Affects queuing space o Complicates pathway connections
Ancillary Facilities (Bus stops, parking)	o Minimize operational impacts o Minimize traffic impediments	o Board/alight passengers at safest point o Assure that pedestrians are visible around parked vehicles
Structures	o Provide adequate laneage	o Provide adequate separation from traffic o Provide physical separation where needed

design "think about the pedestrian" would promote safer and more convenient designs. It is not out of the question for the Green Book to present guidelines for the design of pedestrian facilities, since these most often occur within highway rights-of-way.

RESEARCH NEEDS

There are a number of topics related to integration of pedestrian needs into highway design. Some of these are as follows:

- It is necessary to investigate the functional classification scheme used in the Green Book to determine whether a new scheme could be devised that considers both vehicles and pedestrians.
- Increased roadway costs may result from incorporating features for the pedestrian. A thorough analysis of the life cycle costs and benefits of such actions would be useful in establishing pedestrian-sensitive design standards.
- The modifications of basic highway design features or the incorporation of other features can result in added maintenance costs. There is a need to determine how these features can be designed to minimize maintenance needs and costs.
- A major difficulty in improving streets and highways to better accommodate the pedestrian is the extent of current facilities and established access patterns. The need exists to find effective concepts for the retrofitting of highways to accommodate the pedestrian.

To address the questions posed at the outset of this paper, a structured critique of the highway design process is necessary. The design process must be geared to the primary objective of providing safe and convenient movement for both vehicles and pedestrians.

REFERENCES

1. *A Policy on Geometric Design of Highways and Streets*. American Association of State Highway and Transportation Officials, Washington D.C., 1990.
2. S. A. Smith, K. S. Opiela, L. L. Impett, M. T. Pietrucha, R. L. Knoblauch, and C. Kubat. *Planning and Implementing Pedestrian Facilities in Suburban and Developing Rural Areas*. National Cooperative Highway Research Program Report 294A: Research

Report. TRB, National Research Council, Washington, D.C., June 1987.
3. S. A. Smith, K. S. Opiela, L. L. Impett, M. T. Pietrucha, R. L. Knoblauch, and C. Kubat. *Planning and Implementing Pedestrian Facilities in Suburban and Developing Rural Areas*. National Cooperative Highway Research Program Report 294B: State-of-the-Art Report. TRB, National Research Council, Washington, D.C., June 1987.
4. J. J. Fruin. *Pedestrian Planning and Design*. Metropolitan Association of Urban Designers and Planners, Inc., New York, 1971.
5. *Special Report 209: Highway Capacity Manual*. TRB, National Research Council, Washington, D.C., 1990.
6. *Manual on Uniform Traffic Control Devices for Streets and Highways*. U.S. Department of Transportation, 1988.
7. W. R. McShane and R. P. Roess. *Traffic Engineering*. Prentice-Hall, Englewood Cliffs, N.J., 1990.
8. B. L. Bowman, J. J. Fruin, and C. V. Zegeer. *Planning, Design, and Maintenance of Pedestrian Facilities*. Federal Highway Administration, U.S. Department of Transportation, 1988.
9. B. T. Kulakowski, P. R. Cavanaugh, L. F. Geschwinder, F. Buczek, and P. Pradhan. *Slip Resistant Surfaces Research Project, Volume 1: Technical Report*. Pennsylvania Transportation Institute, University Park, Pa., 1988.
10. C. V. Zegeer. *Synthesis of Safety Research—Pedestrians*. Federal Highway Administration, U.S. Department of Transportation, 1991.
11. M. T. Pietrucha and C. W. Plummer. Design Considerations for Pedestrian Sensitive Geometric Features. In *1992 Compendium of Technical Papers*, Institute of Transportation Engineers, Washington, D.C., 1992.
12. *Traffic Control Device Handbook*. U.S. Department of Transportation, 1983.
13. R. L. Knoblauch, B. H. Tustin, S. A. Smith, and M. T. Pietrucha. *Investigation of Exposure Based Pedestrian Accident Areas: Crosswalks, Sidewalks, Local Streets and Major Arterial*. Report FHA/RD-88/038. Federal Highway Administration, U.S. Department of Transportation, 1988.
14. J. A. Templar. *Provisions for Elderly and Handicapped Pedestrians: Development of Priority Accessible Networks, An Implementation Manual*. Federal Highway Administration, U.S. Department of Transportation, 1980.

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Access Control Issues Related to Urban Arterial Intersection Design

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The 1984 AASHTO Green Book replaced volume-based design with functional design. The Green Book also makes three significant statements relative to intersections: (*a*) driveway terminals are intersections, (*b*) a driveway should not be located within the functional boundary of an intersection, and (*c*) private drives should be designed using the same criteria as the equivalent public street intersections. The third statement implies that there is an equivalence between the intersection of a driveway and a street and the intersection of two public streets—or that such an equivalence needs to be established. The three statements taken collectively imply that there is a hierarchy of intersections. A generalized intersection hierarchy is presented. It is suggested that access control design on major urban arterials should begin with the establishment of optimum spacing criteria for signalized intersections. Minimum functional intersection limits are calculated for various speeds. Basic criteria for unsignalized access are also presented.

The adoption of the philosophy of functional design was a major contribution of the 1984 edition of the AASHTO Green Book (*1*, Chapter 1). However, neither the 1984 nor the 1990 edition effectively addresses access control as an element of design.

The design of urban arterial streets is essentially a matter of designing a hierarchy of intersections, which are connected by relatively short segments of tangents and curves, both horizontal and vertical. Thus the logic of functional design suggests that the urban arterial street design process should begin with the most important intersections and then, in turn, consider intersections that are successively lower in the functional hierarchy.

Major issues relative to access control in urban arterial intersection design include the following:

- Spacing of signalized intersections (private access as well as public streets) so that efficient traffic movement can be achieved on the arterial streets in both peak and off-peak periods;
- Establishment of a functional hierarchy of intersections;
- Determination of the functional boundary of intersections so that an intersection of lower functional classification is not located within the functional boundary of an intersection of higher classification;
- Establishment of comparability between the intersections of two public streets and intersections resulting from the connection of private access drives to public streets;
- Design of intersections (private drives as well as public streets) so that left- and right-turning vehicles do not cause serious interference with through traffic;
- Design of medians and median openings to provide access control at unsignalized intersections, public as well as private; and
- Visibility to the driver of the location and the geometries of each intersection.

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SIGNALIZED ACCESS SPACING

The need to efficiently move high volumes of traffic, especially during peak periods, must be a principal consideration in the planning and design of major urban arterial street systems. The first consideration in the control of access related to urban arterial intersections must then be the selection of an appropriate long and uniform interval for the spacing of signalized intersections. It is essential that high-volume intersections (including signalized private access) conform to the selected spacing or to multiples of this spacing.

The relationship between the speed of progression, cycle length, and signalized intersection spacing is shown in Figure 1, and similar information is presented in Table 1. The shaded areas in Figure 1 indicate the combinations of cycle lengths and desirable speeds for peak and off-peak conditions. Long cycle lengths are commonly used during peak traffic periods to minimize the time lost due to signal phase changes. Also, maximum flow rates are achieved at speeds of 30 to 35 mph (approximately 45 to 55 km/hr). Inspection of Figure 1 and Table 1 shows that a cycle length of 120 sec and a speed of 30 mph (48 km/hr) require a spacing of ½ mi (0.805 km) to achieve optimum progression efficiency.

Given a ½-mi spacing, for example, use of a 60-sec cycle would result in a speed of progression of about 58 mph (95 km/hr) as shown in Figure 2. Undoubtedly, the desired off-peak speed would be less. Use of a slower speed (50 mph or about 80 km/hr) and a 60-sec cycle would result in poor progression efficiency, and platoon movement on the arterial would be interrupted. As shown in Figure 2, a 70-sec cycle should be used for an off-peak speed of 50 mph (80 km/hr). This would maintain efficient progression of the desired speed on the arterial and will increase the delay for left-turning and crossing traffic on the minor street approaches to the arterial. The necessity (or at least the desirability) of providing efficient traffic flow in both the peak and off-peak periods therefore suggests the following for addressing signalized access to major arterial streets.

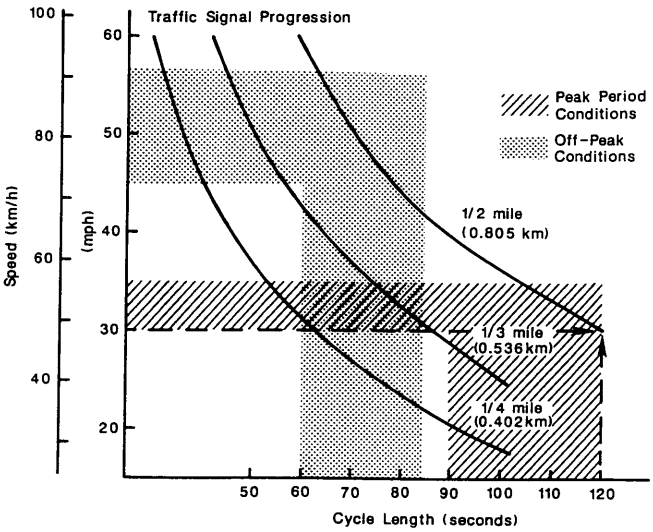
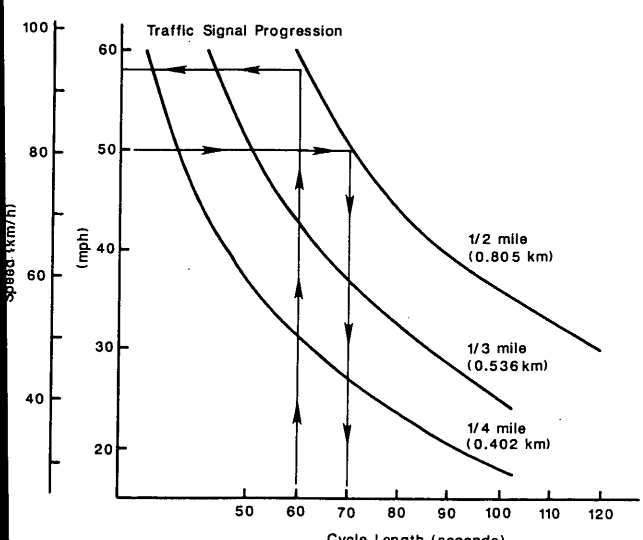


FIGURE 1 Relationships between progression speed cycle length and signal spacing (*2*, Figure 4-13).

TABLE 1 Optimum Signalized Intersection Spacings Needed To Achieve Efficient Traffic Progression at Various Speeds and Cycle Lengths (*3*, p. 29)

Cycle Length (sec)	Speed (mph)						
	25	30	35	40	45	50	55
60	1,100	1,320	1,540	1,760	1,980	2,200	2,430
70	1,280	1,540	1,800	2,050	2,310	2,500	2,820
80	1,470	1,760	2,050	2,350	2,640	2,930	3,220
90	1,630	1,980	2,310	2,640	2,970	3,300	3,630
120	2,200	2,640	3,080	3,520	3,960	4,400	4,840
150*	2,750	3,300	3,850	4,400	4,950	5,500	6,050

* Represents maximum cycle length for actuated signal if all phases are fully used.
One-half mile (2,640 feet) spacing applies where optimum spacing exceeds one-half mile.



IGURE 2 Selection of peak-period cycle length once a niform signalized intersection spacing has been established /, Chapter 2).

• Adopt a signalized interval that will maximize efficiency at 30 mph (45 km/hr) with the anticipated future peak-period cycle lengths.

• Use the comprehensive urban plan and other development management tools to implement the desired signalized intersection spacing.

FUNCTIONAL HIERARCHY

The AASHTO Green Book recognizes that a functionally designed circulation system provides for the following distinct travel states: termination, access, collection, distribution, primary movement, and transition (*1*, pp. 1–3; *5*, pp. 1–3). It also indicates that, according to functional design concepts, each trip stage should be handled by a separate facility and that "the failure to recognize and accommodate by suitable design each of the different stages of the movement hierarchy is a prominent cause of highway obsolescence" (*1*, p. 2; *5*, p. 3). The AASHTO policy also indicates that the same principles of design should be applied to access serving development adjacent to arterials and collector streets.

Thus functional design principles recognize the following:

• All trips begin at the origin and end with termination at the destination end. However, all trips do not necessarily involve all the trip stages (i.e., primary movement).

• The design of each facility should be based on the degree to which it is to serve the functions of movement versus access.

As shown in Figure 3, the three principal functional classes are divided into various general design classes. These classes can be further subdivided into typical design groups (i.e., major arterials with at-grade intersections can be subdivided into typical cross sections such as six-lane divided and four-lane divided, which can be further subdivided by more detailed design elements such as median width/design, etc.).

The fact that driveways create intersections with the public street system is recognized by AASHTO (*1*, p. 888; *5*, p. 841). Though not clearly stated, the recognition of transition as a

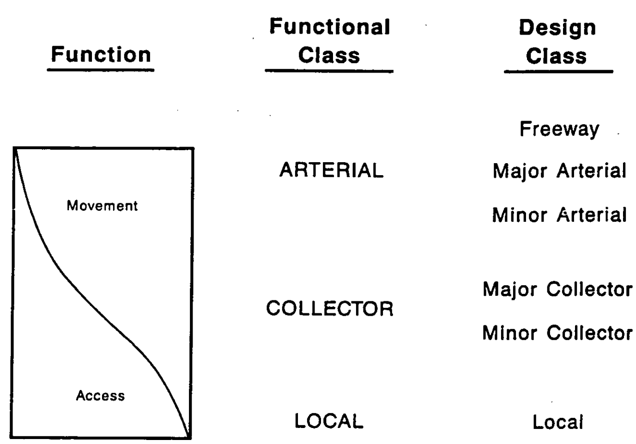


FIGURE 3 Hierarchy of facilities in a functionally designed street system.

separate trip stage combined with the concept of functional classification (the degree to which a given facility is to accommodate movement versus access) implies that there is a hierarchy in the provision of transitions between facilities intended to accommodate different degrees of access versus movement and that there is (or should be) a hierarchy in the design of intersections.

As shown in Figure 4, transition from or to a facility of lower functional design should be limited to a facility of the next higher design, that is, a local street should intersect with a minor collector or in some cases a major collector in carefully designed residential subdivisions. Major collectors should only intersect with an arterial, and only major at-grade arterials should interconnect with freeway-type facilities. Private access drives (the termination facilities) serving individual dwellings should intersect with local streets and minor residential collectors only. This concept of design is well recognized in the design of modern limited-access subdivisions, and municipal ordinances commonly prohibit private residential driveways and local streets from intersecting with arterials. However, this principle is not as well recognized in the provision of access between major streets and adjacent nonresidential development.

Though implied by functional design principles, there is very little literature addressing the hierarchy of intersections of public streets and their private access or on-site circulation equivalents. A suggested generalized hierarchy is given in Table 2. Two or more typical designs might be developed for each depending on the conditions commonly encountered by an agency. For example, the intersection of a major arterial and a major collector where the collector serves a residential development will experience different traffic conditions than an intersection of the same major arterial and a major collector servicing retail, office, or industrial development.

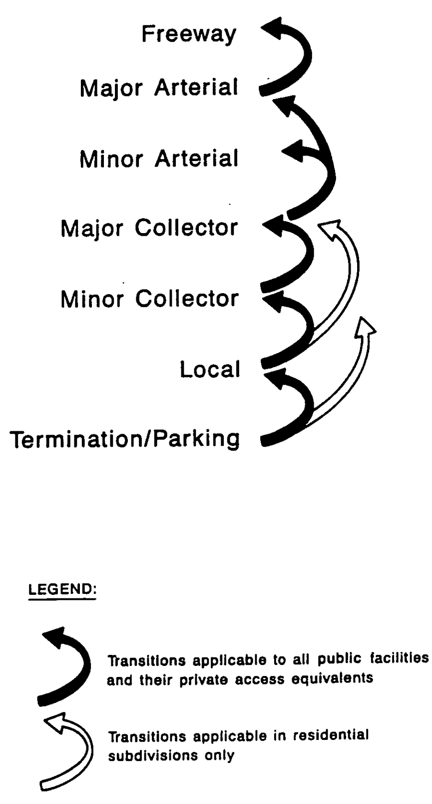


FIGURE 4 Functional hierarchy of intersections.

TABLE 2 Generalized Hierarchy of Intersections

High	Freeway Main Lanes - Ramp
	Major Arterial - Freeway Ramp
	Major Arterial -
	Major Arterial - Minor Arterial
	Major Arterial - Major Collector
	Minor Arterial - Major Collector
	Major Collector - Major Collector
	Minor Collector - Local
Low	Local - Residential Drive/Parking Place

Consequently, there should be at least two different typical designs for major arterial–major collector intersections. Moreover, there is an increasing awareness that some major arterials are of regional significance, whereas other major arterials are of significance to a portion of a large urban region. Thus there should be least five typical major arterial–major arterial intersections. In order of decreasing hierarchical importance these are (*a*) six-lane regional to six-lane regional, (*b*) six-lane subregional to six-lane subregional, (*c*) six-lane regional to four-lane subregional, (*d*) six-lane subregional to four-lane subregional, and (*e*) four-lane subregional to four-lane subregional. This also means that the hierarchy of intersections is much more complex than the generalized hierarchy presented in Table 2.

FUNCTIONAL BOUNDARY

AASHTO specifically states that "a driveway should not be located within the functional boundary of an intersection" (*1*, p. 888; *5*, p. 841). Whereas AASHTO does not present guidelines as to the size of the functional area of an intersection, logic indicates that it must be much larger than the physical area (see Figure 5). Logic also suggests that the functional area should be composed of the distance traveled during the PIEV time plus the distance required to move laterally and come to a stop plus any required storage length (see Figure 6). The minimum maneuver distance assumes that the driver is in the proper lane and only needs to move laterally into a right-turn bay (as shown) or a left-turn bay. Parameters that must be evaluated in the determination of maneuver distance include the following:

- d_1: The perception-reaction time required by the driver depends on driver characteristics. It may be assumed that for motorists frequently using the street this is a little more than the time required for preparing to brake. Strangers may not be in the proper lane to execute the maneuver and could require several seconds to react.
- d_2: Braking while moving laterally is a more complex maneuver than braking alone, perhaps one-half the deceleration rate used in d_3. Lateral movement under urban condi-

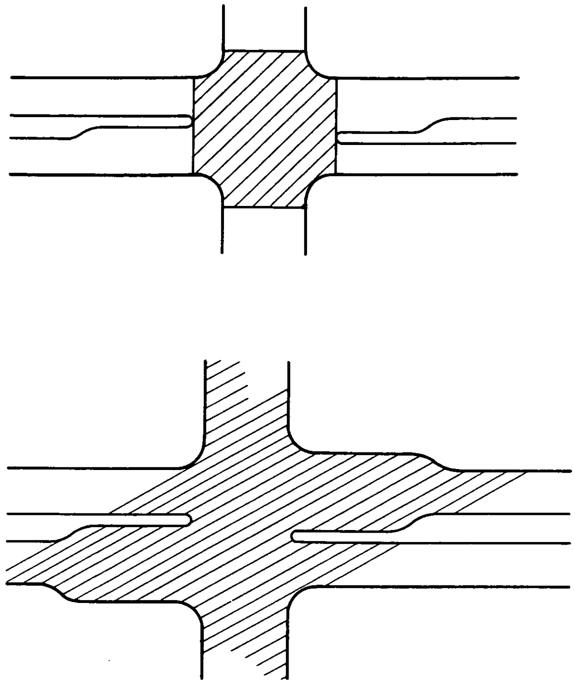


FIGURE 5 Intersection area: *top*, defined by physical area; *bottom*, defined by functional area (*2*, Figure 4-16).

tions is commonly assumed to be 4 ft/sec (1.2 m/sec). At low deceleration rates [less than about 7 ft/sec² (2 m/sec²)], the driver will have shifted laterally sufficiently far that a following vehicle can pass without encroaching on the adjacent lane before a speed differential of 10 mph (16 km/hr) occurs. At higher deceleration rates [more than about 7 ft/sec² (2 m/sec²)], the speed differential will exceed 10 mph (16 km/hr) before the turning vehicle clears the through-traffic lane.

- d_3: Deceleration after moving laterally into the turn bay should be a rate that will be used by most drivers. Studies (*6,7*) have found that most drivers (85 percent) will use a deceleration rate of 6 ft/sec² (1.8 m/sec²) or more; only about 50 percent can be expected to accept a rate of 9 ft/sec² (2.7 m/sec²) or greater.
- d_4: Queue storage should be of sufficient length to accommodate all turning vehicles most of the time.

Table 3 presents maneuver distances and total distances (maneuver plus PIEV distance, but excluding storage) for the conditions indicated in the footnotes. These distances represent the minimum functional length of an approach to an intersection since they exclude storage.

As indicated in Figure 6, the physical length of a turn bay excludes the distance traveled during the perception-reaction time. The difference in the maneuver distance required for peak and off-peak speeds will provide some storage. For example, using the desirable values, an off-peak speed of 55 mph (90 km/hr) and a peak speed of 30 mph (50 km/hr), a storage of about 450 ft (135 m) is built in. At 25 ft (7.7 m) per vehicle, measured front bumper to front bumper, a queue of about 18 cars can be accommodated. This will generally be sufficient to provide the necessary right-turn storage on arterial approaches at the intersections with minor arterials and major collectors serving residential areas. At high-volume intersections, the functional limits are commonly controlled by peak-period conditions since peak-period maneuver distance plus storage is longer than the maneuver distance plus storage needed in the off-peak. Consequently, the functional boundaries will be greater than the distances given in Table 3 or those developed from similar analyses.

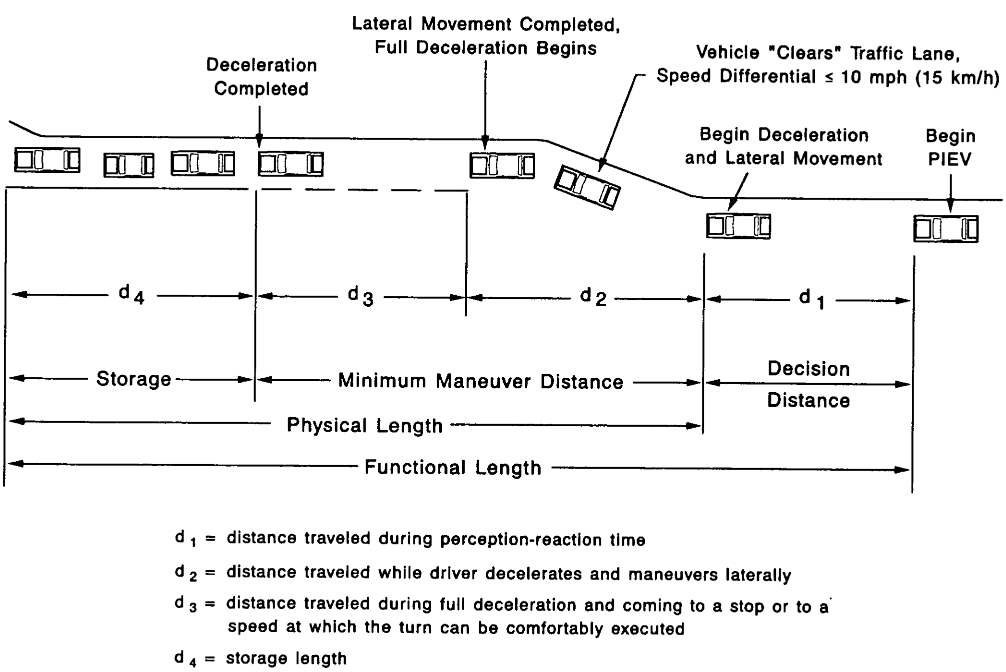


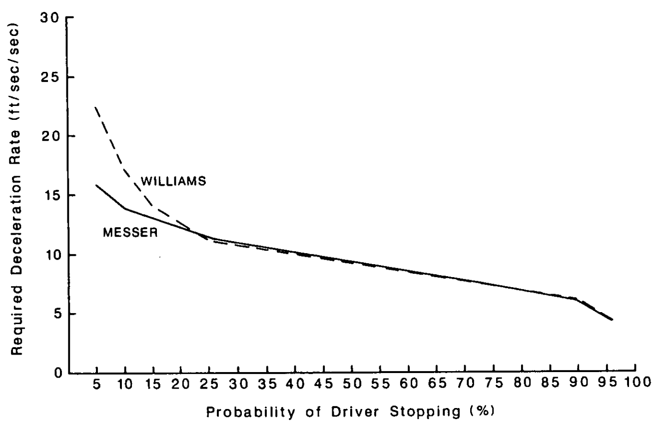
FIGURE 6 Elements of the functional area of an intersection.

TABLE 3 Calculated Upstream Maneuver Distances

	Minimum Maneuver Distance [1] in Feet (Metres)			
	Desirable Conditions [2]		Limiting Conditions [3]	
Speed mph (km/h) [4]	Deceleration[5]	Total[6]	Deceleration	Total[6]
30 (50)	225 (70)	325 (100)	170 (50)	215 (65)
35 (55)	295 (90)	425 (130)	220 (65)	270 (80)
40 (65)	375 (115)	525 (160)	275 (85)	335 (700)
45 (70)	465 (140)	630 (190)	340 (105)	405 (125)
50 (80)	565 (170)	750 (230)	410 (125)	480 (145)
55 (90)	675 (205)	875 (265)	485 (150)	565 (170)
60 (95)	785 (240)	1005 (305)	565 (170)	655 (200)

Source: Calculations by author.

(1) All values rounded to nearest 5 feet (5 metres).
(2) 2.5 second perception-reaction time; 3.5 fps² (1.1 mps²) average deceleration while moving laterally into turn bay and an average 6 fps² (1.8 mps²) deceleration thereafter 10 mph (15 km/h) speed differential.
(3) 1.0 second perception-reaction time; 4.5 fps² (1.4 mps²) deceleration while moving laterally into turn bay and an average 9.0 fps² (2.7 mps²) deceleration thereafter; 10 mph (16 km/h) speed differential
(4) Nearest 5 km/h for design
(5) Distance to decelerate from speed to a stop while maneuvering laterally into a left or right-turn bay.
(6) Deceleration distance plus distance traveled in perception-reaction time.



OBSERVED DECELERATION RATES

The inability to forecast turn volumes with any degree of reliability presents an additional issue in access control relative to urban arterial intersection design. This problem can be addressed by providing flexibility in the design and adoption of appropriate state access regulations and local ordinances. Design can provide flexibility by including dual left turns on all approaches of major arterial intersections. The possibility that it may be necessary to lengthen a turn bay (especially a left-turn bay) should also be considered in the event that demand turns out to exceed the projected turn volume. A major issue is that most states and most local governments have not enacted appropriate legislation or ordinances and standards necessary for implementation of access management of high-speed, high-volume arterials.

The placement of driveways within the functional boundary of an intersection may be expected to have an effect on intersection capacity as well as to increase accident rates. The research by McCoy and Heimann (*8*) suggests that the corner clearance required to diminish the effect on capacity is less than the maneuver distance. Additional research on the effect of corner clearance on intersection capacity over a range of speeds and design conditions is needed.

LIMITATION OF SPEED DIFFERENTIAL

It has long been recognized that a high speed differential between vehicles in the traffic stream is a major factor in traffic accidents (*9*). Research has shown that the forward speed of a vehicle making a turn at an intersection was very slow for all reasonable combinations of curb return radii and throat widths (*2*). As shown in Figure 7, the forward speed of the vehicle is 9 to 14 mph (14 to 22 km/hr). The speed vector parallel to the through traffic lane is only about 1.5 to 2.5 mph (2.4 to 4.0 km/hr). Thus the speed differential between a turning vehicle and through traffic is essentially the speed of traffic on the through lanes. A turn bay is therefore essential if the speed differential between turning vehicles and through traffic is to be limited to some reasonable magnitude.

The minimum physical length of a turn bay, exclusive of storage, for different speeds is given by the deceleration distances in Table 3. Distances similar to those for deceleration (the limiting conditions in Table 3) are included in the New Mexico regulations (*10*, Table 6), for example.

Dual left turn bays, or the provision for future dual left turn bays, should be provided at all major arterial–to–major arterial intersections. This will require a median of at least 30 ft (9 m). As in the case shown in Figure 8, the dual left-turn bay should be striped as a single left-turn bay until volumes warrant dual left operation.

The provision of a dual-left on the arterial street at arterial-collector intersections can be used to facilitate U-turns as shown in Figure 9. This practice provides access to adjacent properties without the provision for a median break.

The taper length in rural design is commonly a ratio that is a function of the design speed. It is suggested herein that the taper for urban arterial design should be a standard length. It is also suggested that the taper for a single left-turn and for a right-turn bay should be 120 ft (35 m) or less. This is only slightly shorter than the distance used by a driver moving laterally the width of one traffic lane at peak-period speed [(1.47) (30 mph) × (12 ft/4 ft/sec) = 132 ft]. This follows from the fact that urban arterials have two design conditions, namely peak and off-peak. With dual left-turn bays, a taper of 180 or 150 ft (55 or 45 m) is suggested. These taper lengths are slightly more abrupt than the natural trajectory used by drivers at peak-period speeds and thus help communicate that the increased cross section is an auxiliary lane (turn bay) rather than an additional lane.

COMPARABILITY OF PUBLIC AND PRIVATE ACCESS

AASHTO makes the following significant statement: "Driveway terminals are in effect at-grade intersections and should be designed consistent with the intended use" (*5*, p. 841). To implement design based on this principle, it is necessary to establish an equivalence between public streets and driveways. Such analogies are presented in Table 4.

UNSIGNALIZED INTERSECTIONS

Medial and marginal access to urban arterial streets may be provided as long as it does not jeopardize the primary (move-

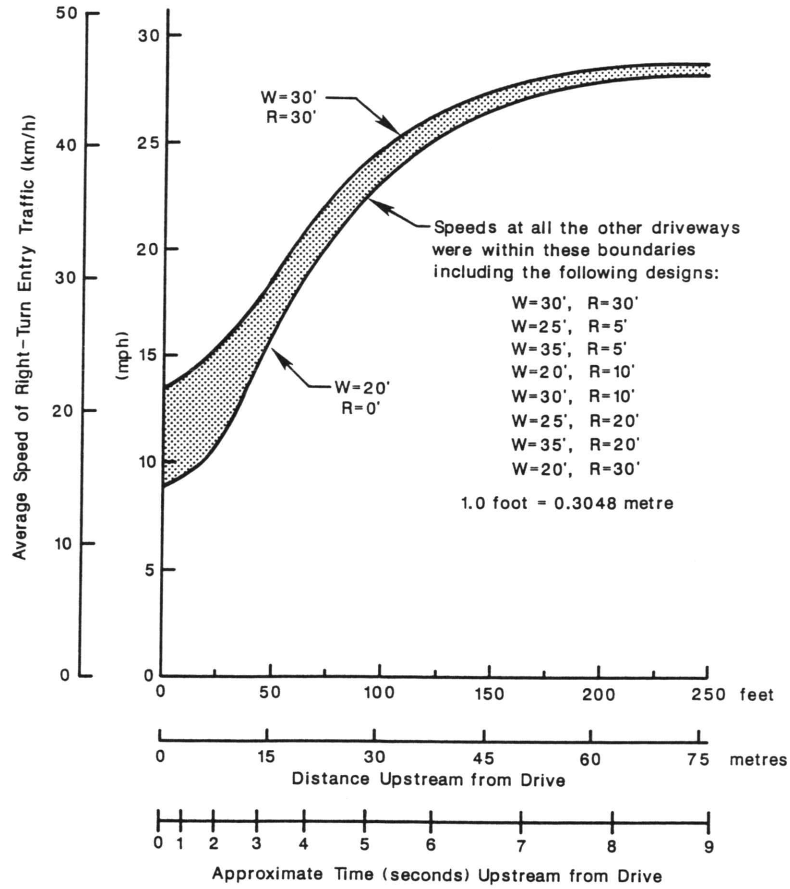


FIGURE 7 Vehicle speed as a function of driveway geometry (*2*, Figure 5-29).

FIGURE 8 When this roadway (Powers Boulevard, Colorado Springs, Colorado) was reconstructed as a multilane major arterial, long dual left-turn lanes (970 ft, including taper) were provided. The bay is currently striped to operate as a single left-turn lane until volumes require a dual left. Such a length will accommodate maneuvers at a 55-mph off-peak speed and ensure availability of maneuver plus storage space when the area fully develops.

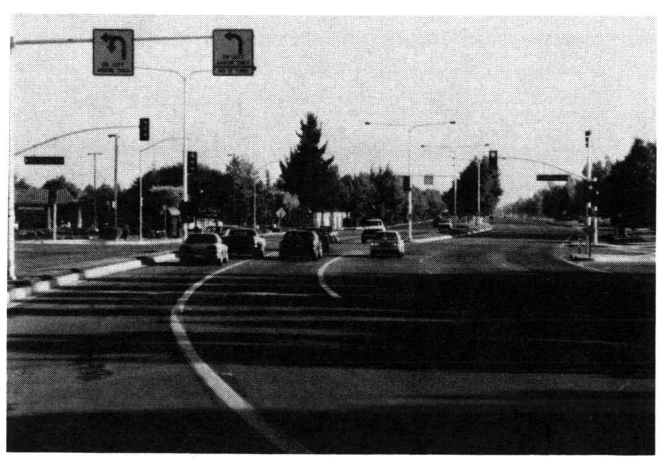


FIGURE 9 Allowing U-turns at low signalized intersections where the cross street has relatively low approach volumes (arterial-collector intersections) facilitates access to properties fronting on major street without the provision of median breaks. A 30-ft median is desirable for such designs. This provides for two turn lanes and a median nose of at least 6 ft.

TABLE 4 Comparability Between Public Streets and Circulation Elements of Private Development (2, p. 85)

Public Street	Private
Local	Aisle in parking lot; Driveway of convenience grocery
Minor Collector	Perimeter road in shopping center; Access drive of small commercial center (> 200,000 ft²)
Major Arterial	Access drive of a large retail development (> 1,000,000 ft²); Access drive of very large commercial or mixed use development (> 2,000,000 ft²)

ment) function of the arterial. Identification of the frontage where access can be provided is essentially a problem of determining the functional boundary of the adjacent intersections. It is suggested that the problem be addressed in two steps. First, establish the frontage along the arterial where access can be provided without interfering with adjacent intersections, especially signalized intersections. Second, identify the specific location and design of the access drive in conjunction with the design of adjacent property.

Figure 10 shows the region along the property frontage at which direct access might be permitted on the basis of the AASHTO policy that a driveway should not be situated within the functional boundary of an intersection. In locating the driveway it should be recognized that a turn bay is essential to limit the speed differential between a turning vehicle and through traffic. Therefore, the frontage on which the driveway throat can be located is shorter than the distance between the functional intersection boundaries.

Consolidation of access is another practice that minimizes access to arterial streets and increases corner clearances. Where the corner property is a separate parcel, a shared access point may be provided in the subdivision process. An example of this practice is shown in Figure 11. Where the adjacent property has already been subdivided, the adjacent property owners might be encouraged to consolidate access in a similar manner.

Medial access at unsignalized intersections should be designed to permit specific left turns (i.e., left-turn ingress or left-turn egress), and crossing movements should be positively eliminated by the design of the median opening. An example of a good design is shown in Figure 12.

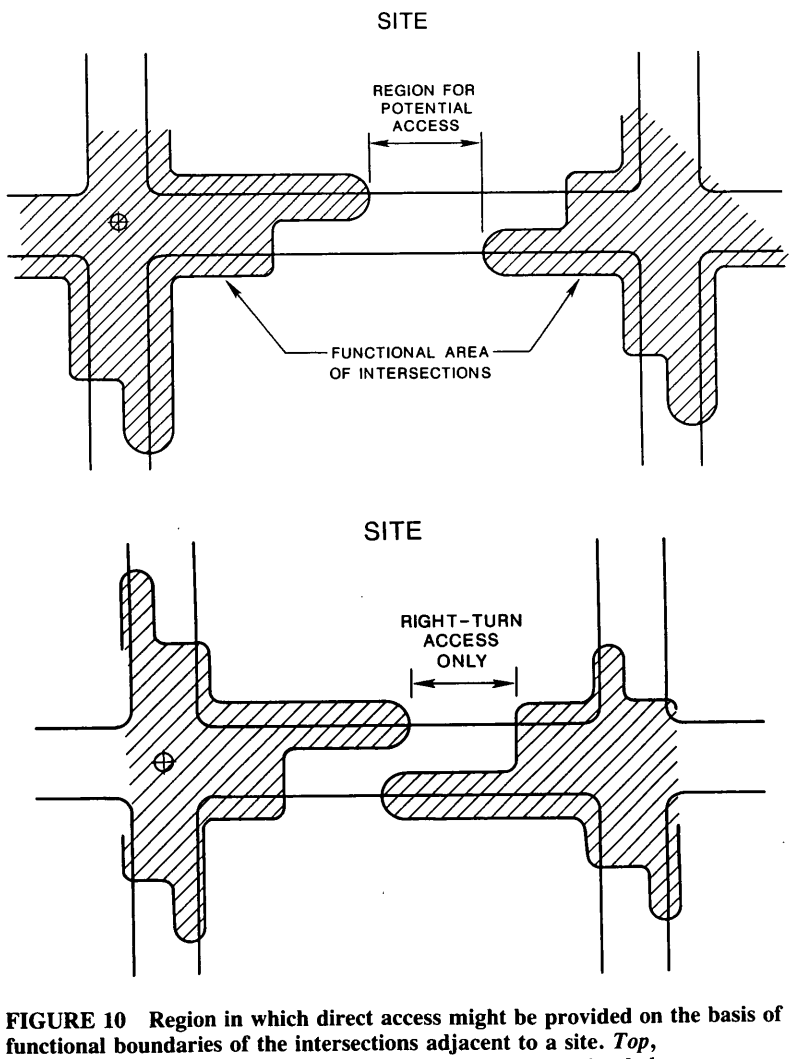
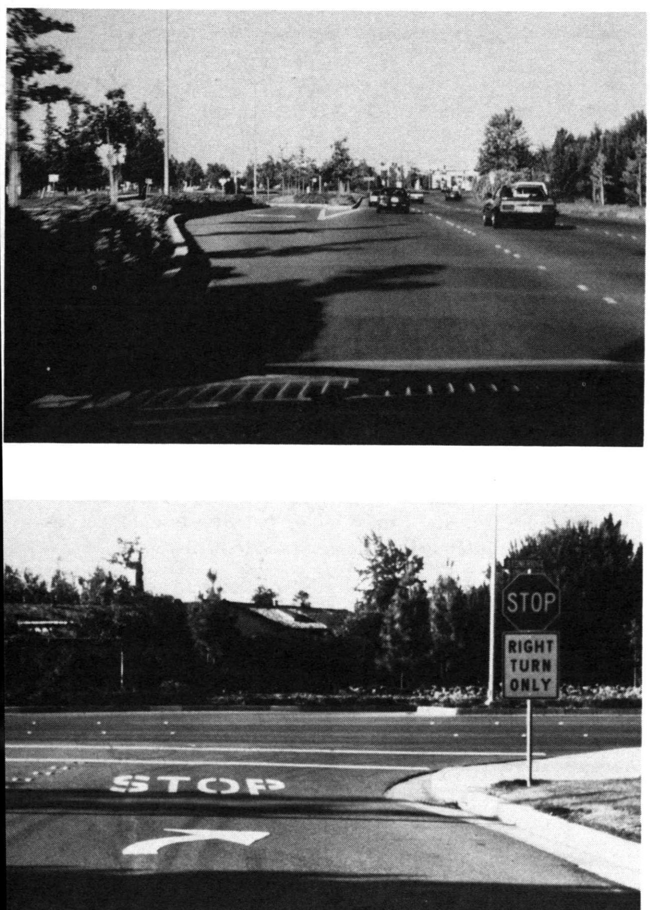


FIGURE 10 Region in which direct access might be provided on the basis of functional boundaries of the intersections adjacent to a site. *Top*, condition where left- and right-turn access might be permitted; *bottom*, condition where right-turn access only should be permitted.

FIGURE 11 Sharing of access will reduce the number of access drives and increase their spacing, or parcels with smaller frontage can be developed given a driveway spacing or corner clearance. In this example, a fast-food restaurant shares access with a 200,000-ft² shopping center. The centerline of the access drive is located on the property line. A reciprocal easement runs the length of the restaurant property. This is the only direct access to the restaurant.



IGURE 12 Examples of landscaping design to provide good elineation of a median break designed for left-turn egress only. *op*, driver's view on the approach to the left-turn bay; *bottom*, iew of the median treatment from the intersecting access.

VISIBILITY

The driver must be able to determine the location and the geometries of the intersection sufficiently far in advance so as not to be surprised either by the intersection itself or by other drivers making ingress or egress maneuvers. Greenberg (*11*) points out a serious issue in the use of the AASHTO Green Book, namely, that the sight distance criteria for minimum length of crest vertical curve is less than the intersection sight distance. Consequently, extreme care must be taken to ensure that an intersection (especially an unsignalized intersection with a public street or a private access drive) is not located beyond the crest of a vertical curve. Similarly, when the intersection itself is beyond the crest, the driver should be able to see a substantial portion of a left- or right-turn bay on the approach to a crest vertical curve.

Landscaping is essential to provide identification of the location of the median opening, the geometries of the intersection for the movement permitted, and clear definition as to which maneuvers are not permitted. Long left-turn bay tapers also need to be avoided to prevent misleading through traffic. Landscaping is also desirable to delineate the median nose where long left-turn bays are provided.

Delineation of marginal access at unsignalized intersections (public streets as well as private driveways) is also needed to inform drivers of the location and geometries. Visibility problems arise when adjacent development is allowed to pave to the back of the curb, the sidewalk is located immediately adjacent to the back of the curb, the intersection area is partly illuminated, and the entrance sign is placed downstream from the access drive.

The maneuver distances associated with high-speed urban arterials result in very long median noses. To enhance their visibility, landscaping should be provided, as shown in Figure 13.

FIGURE 13 A long median nose will result where a left-turn bay is designed to accommodate high off-peak speeds appropriate to major arterial streets or where long queues must be stored during peak periods. Landscaping helps delineate and enhance the visibility of these intersections. A width of at least 6 ft (preferably at least 8 ft), face-to-face of curb, is needed for pedestrian refuge as well as to accommodate landscaping. In this example, the planter boxes contain flowers, which represent a high-maintenance item. Selection of perennial plants will reduce the maintenance cost.

Figure 14 shows an example where development adjacent to the through traffic lanes was made in such a manner that the access drives are difficult to identify and locate. Development adjacent to a major street should not be permitted to pave the right-of-way. Also, sidewalks should not be located adjacent to the back of the curb. Rather, a landscape area should be required as part of the development adjacent to major streets. The area between the back of curb and the right-of-way line should be incorporated into the landscape scheme. The contrast between the paving and the landscaping will assist the driver in identifying the location and geometries of the access drive.

Median landscaping is also desirable to provide delineation of the median and inform drivers that left turns are prohibited by the median design, as in the example shown in Figure 15.

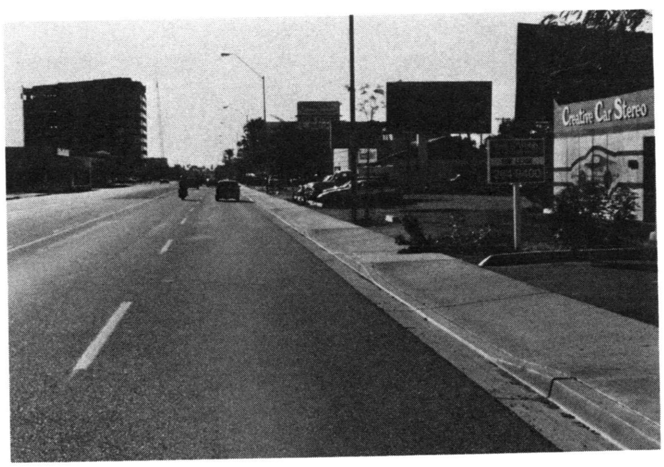


FIGURE 14 Placing sidewalks immediately adjacent to the bank of curb and allowing the owners of the adjacent properties to pave the right-of-way result in extremely poor delineation of the driveway location and geometry. A landscaped buffer in addition to the right-of-way should be required to enhance driveway identification.

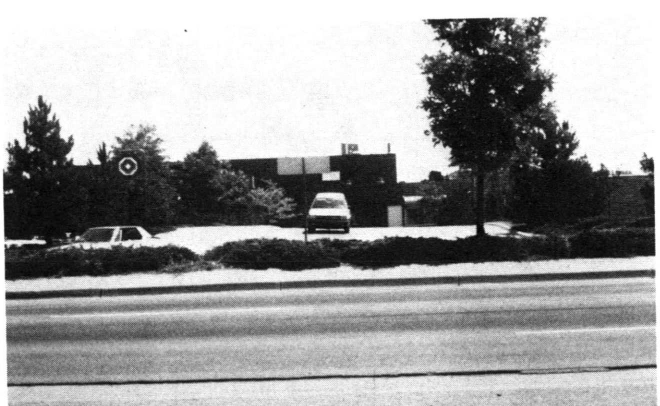


FIGURE 15 This section of US-36 in Boulder, Colorado, was reconstructed with a nontraversable, landscaped median. Movements are now restricted to right turns only. The landscaping adds aesthetics as well as function.

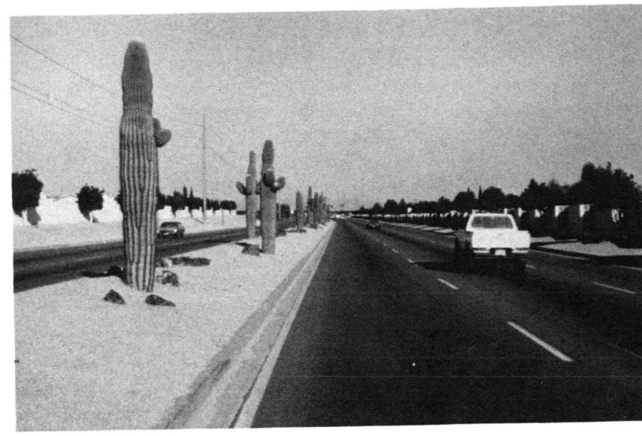


FIGURE 16 US-60/69 in Sun City, Arizona, is an example of medial landscaping using native materials resulting in low maintenance cost and providing good visibility.

FIGURE 17 Signing must be considered part of the design of intersections that provide access to development. The sign is located on the far side of a divided access drive to a regional shopping center. The problem is compounded by the intersection's location on a horizontal curve to the right. These two elements combine to lead drivers past the ingress drive. Many drivers miss the drive at night and stop in the traffic lane. Of those who stop, most either back up to enter on the ingress side or enter using the egress side of the drive.

Appropriate choice of landscaping materials helps control maintenance costs. Such an example is shown in Figure 16.

The location of signs is also an important feature of the design of intersections serving private development adjacent to major streets. Drivers approaching these intersections use the signs to help identify the location of the driveway. Special care must be taken where marginal access is located on a horizontal curve, such as the example shown in Figure 17.

REFERENCES

1. *A Policy on the Geometric Design of Highways and Streets*. American Association of State Highway and Transportation Officials, Washington, D.C., 1984.

2. V. G. Stover and F. J. Koepke. *Transportation and Land Development*. Institute of Transportation Engineers, Prentice Hall, Inc., 1988.
3. *Access and Spacing Standards for State Highways*. Urbitran Associates, Inc., Feb. 1990.
4. V. G. Stover et al. *City Street Design*. Short course notes. Texas Transportation Institute and Texas Engineering Extension Service, Dec. 1990.
5. *A Policy on the Geometric Design of Highways and Streets*. American Association of State Highway and Transportation Officials, Washington, D.C., 1990.
6. M. S. Chang, C. J. Messer, and J. Santigo. Timing Traffic Signal Change Intervals Based on Driver Behavior. In *Transportation Research Record 1027*, TRB, National Research Council, Washington, D.C., 1985.
7. W. L. Williams. Driveway Behavior During the Yellow Time Interval. In *Transportation Research Record 664*, TRB, National Research Council, Washington, D.C., 1977.
8. P. T. McCoy and J. E. Heimann. Effect of Driveway Traffic on Saturation Flow Rates of Signalized Intersections. *ITE Journal*, Feb. 1990.
9. D. Solomon. Accidents on Main Rural Roads Related to Speed, Driver and Vehicle. *Public Roads*, July 1964.
10. *Regulations for Driveways and Medial Openings on Non-Access Controlled Highways*. New Mexico State Highway and Transportation Department, June 9, 1989.
11. F. M. Greenberg. Design of Crest Vertical Curves: Proceed With Caution! *Public Works*, April 1986, pp. 42–43.

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Intersection Design Considerations To Accommodate Large Trucks

JOHN M. MASON, JR., KAY FITZPATRICK, DOUGLAS W. HARWOOD, AND JUSTIN TRUE

The geometric and operational considerations of large trucks at intersections are addressed. The discussion summarizes the related findings of an FHWA study of truck characteristics for use in highway design and operations. Also cited are the implications of the Turner truck proposal, comparisons with Canadian Interprovincial trucks, and the preliminary results of recent research on AASHTO intersection sight distance procedures. A brief review of channelization requirements of large trucks, the effects large trucks have on intersection capacity and traffic signalization, and advance warning sign placement considering truck stopping distance and deceleration rates is provided.

At-grade intersections vary greatly in type, size, and function. The geometric features of an intersection are influenced by four basic elements: human factors, traffic characteristics, physical elements, and economic factors (*1*). This paper addresses the geometric and operational considerations of large trucks at intersections. The discussion draws primarily on a recently completed study by Harwood et al. that identified, evaluated, and assessed large truck characteristics for use in highway design (*2*). Several other publications are referenced that provide guidance on related issues not fully addressed in the AASHTO 1990 Green Book (GB90) policy (*1*). Among the more salient documents are the Turner proposal, Canadian Interprovincial truck regulations, and intersection sight distance procedures (*3–5*).

The truck characteristics section of this paper demonstrates the implications of a changing commercial vehicle fleet. The findings of the TRB special study of the Turner truck proposal highlight the effects of the prototypes on future at-grade intersection designs. A comparison of Canadian truck weights and dimensions with the AASHTO GB90 policy is also provided. The phenomenon known as offtracking, as influenced by pavement cross-slope, is explained for both low-speed and high-speed maneuvers. A comparison of truck offtracking performance considering turning angle and radius is presented for specific truck-trailer combinations.

TRUCK PHYSICAL CHARACTERISTICS

The size and operating characteristics of vehicles that are expected to use an at-grade intersection are primary considerations in its design. Vehicle characteristics that affect intersection design are physical characteristics such as width, length, number of axles, distance between axles, and number of articulation points. Vehicle operating characteristics such as offtracking, acceleration, and deceleration also influence the overall intersection design. For highway design purposes, trucks are generally classified as single-unit or straight trucks, straight trucks with trailer, tractor-semitrailers, tractor-semitrailer–full trailer (commonly referred to as a double or twin trailer truck), and tractor-semitrailer–full trailer–full trailer (commonly referred to as a triple).

Trends in Truck Size and Weight

As the demand for movement of freight by trucks has increased, trucks have become larger and heavier. For example, in 1927 the median weight limit for combination trucks was less than 40,000 lb; the median weight limit rose to 48,000 lb in 1940, 72,000 lb in 1964, and 80,000 lb in 1982 (*6*). In 1949, the median length limit was 50 ft for a combination truck. This had increased to 55 ft by 1964 and 65 ft by 1982 (*6*). Width limits have been more stable, with most states setting a maximum width of 96 in. until 1982, when federal legislation increased the allowable width to 102 in.

Until 1956, truck size and weight regulations were established by state and local jurisdictions. However, the Federal Aid Highway Act of 1956 established maximum limits on Interstate highways for vehicle width (96 in.) and gross weight (73,280 lb). Federal legislation in 1975 allowed vehicles up to 80,000 lb to operate on the Interstate system.

The 1982 Surface Transportation Assistance Act (STAA) expanded the role of the federal government in the regulation of truck size and weight. For the first time, the federal government required states to change existing truck size and weight limits in the interest of national uniformity.

The act applies to trucks operating on the National Truck Network. This network includes the Interstate system and certain other highways for a total mileage of about 183,000 mi. The act requires the states to allow trucks of the following dimensions and number of trailers to operate on the National Truck Network. Provisions of the act that affect highway design are as follows:

- Trailers up to 48 ft long in a tractor-semitrailer combination cannot be prohibited by the states. Grandfather provisions in the act allow the continued operation of trailers larger than 48 ft in states where they were legal before passage of the STAA.

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- Trailers up to 28 ft long in a tractor-semitrailer–full trailer (double) combination cannot be prohibited by the states. Again, a grandfather provision allows the continued operation of longer trailers if they were legal in a state before the passage of the STAA.
- The width limit was increased from 96 to 102 in.

The STAA is important from a highway design standpoint because longer and wider trucks have become more common and need to be accounted for in the design of at-grade intersections.

Current Design Vehicles

The GB90 includes 15 design vehicles: 1 passenger car, 2 buses, 4 types of recreational vehicles, and 8 trucks. The dimensions of these vehicles for use in design can be found in Table II-1 of GB90 (*1*). The GB90 recommends that the "design vehicle" be likely to use the facility with considerable frequency or to have special characteristics. For example, the turnpike double is unique because it has an overall length of about 118 ft and in a low-speed turn offtracks about 25 ft. This is in contrast to a tractor-semitrailer with a 48-ft trailer, which has an overall length of about 65 ft and offtracks about 17.3 ft.

Turner Proposal

TRB recently completed a comprehensive study of the Turner truck proposal (*3*). This proposal, suggested in 1984 by former Federal Highway Administrator Francis C. Turner, recommends an approach for reducing pavement wear while increasing truck productivity. Pavement wear would be reduced by lower axle loads (more axles per truck), and truck productivity would be improved by increasing gross vehicle weight.

The TRB study estimated the costs and benefits of the Turner proposal in the areas of truck productivity, safety, traffic, bridges, and pavements. In conducting the study, four prototype trucks were used. Three baseline trucks were used for comparison purposes. The four prototype trucks are described in Table 1.

As previously discussed, one characteristic that affects at-grade intersection design is low-speed offtracking. Table 2 gives the low-speed offtracking characteristics of the four prototype trucks and the three baseline trucks used in the TRB study (*3*). An examination of Table 2 shows that the nine-axle B-train double offtracks about 3.5 ft more than a conventional five-axle tractor-semitrailer with a 45-ft trailer.

A nine-axle B-train double would have difficulty in maneuvering at an at-grade intersection designed to accommodate a five-axle trailer-semitrailer with a 45-ft trailer. The 11-axle double trailer combination prototype was excluded from the study because it was determined that this truck type would require costly bridge improvements.

Canadian Truck Limits

In 1988, the Canadian Council of Ministers of Transportation and Highway Safety agreed to a memorandum of understanding establishing uniform truck weights and dimensions (*4*). The approved weights and dimensions apply only to trucks engaged in interprovincial commerce. The following four categories of trucks are covered: tractor-semitrailer, A-train, B-train, and C-train. Figure 1 shows the allowable dimensions for these four categories of trucks. The dimensions shown are direct conversions from metric to the nearest tenth of a foot.

TABLE 1 Four Turner Prototype Trucks (*3*)

Characteristic	Seven-Axle Tractor-Semitrailer	Nine-Axle Double	11-Axle Double	Nine-Axle B-Train Double
Dimensions[a]				
Overall length (ft)	60	81	81	81
Trailer length (ft)	48	33	33	33
Trailer axle width (in.)	102	102	102	102
Weight[b] (lb thousands)				
Gross if limited by axle weights only[c]	91	111	141	111
Gross if also limited by federal bridge formula	91	111	127	111
Payload	60	73	101 or 87	73
Tractor	Conventional	Conventional	Conventional	Conventional
Trailer	Flatbed, van, bulk	Flatbed, van, bulk	Flatbed, van, bulk	Flatbed, van, bulk
Tires[d] (no. on loaded axles)	1 or 2	1 or 2	1 or 2	1 or 2
Dolly	None	Single or double drawbar	Single or double drawbar	None
Suspension[e]				

[a]For evaluating impacts, axle spacings and kingpin and fifth wheel positions must also be specified.
[b]Empty weights and axle weight distributions must also be specified.
[c]Single axle, 15,000 lb; tandem axles, 25,000 lb; triaxles, 40,000 lb; steering axle practical limit, 11,000 lb.
[d]Tire size and pressure must also be specified.
[e]Suspension types, use of lift axles, use of steerable axles, use of belly axles, or other arrangements within multiaxle groups must be specified.

TABLE 2 Offtracking Characteristics of Turner Prototype Trucks and Baseline Trucks (3)

Characteristic	Polarity of Measure	Baseline Vehicle: Five-Axle Tractor-Semitrailer[a]	Baseline Vehicle: Five-Axle Twin Trailer[b]	Baseline Vehicle: Nine-Axle Turnpike Double[c]	Prototype Vehicle: Seven-Axle Tractor-Semitrailer[d]	Prototype Vehicle: Nine-Axle B-Train Double[e]	Prototype Vehicle: Nine-Axle Double[e]	Prototype Vehicle: Eleven-Axle Double[e]
Low-speed offtracking (ft)	Lower values better	15.36	14.27	25.11	15.81	18.87	15.96	13.17

[a]With 45-ft semitrailer.
[b]With 28-ft trailers.
[c]With 45-ft trailers.
[d]With 48-ft semitrailer.
[e]With 33-ft trailers.

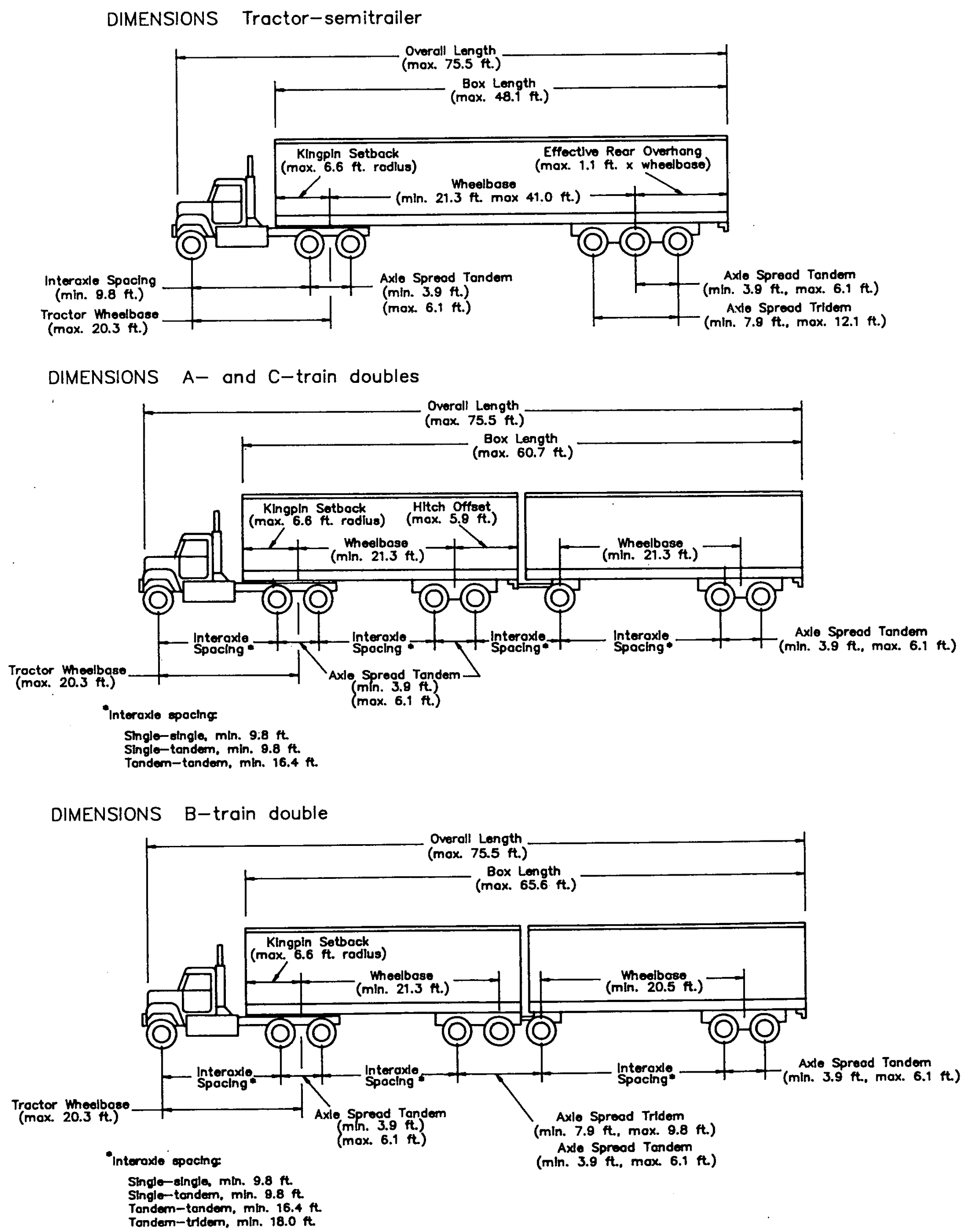


FIGURE 1 Canadian Interprovincial truck dimensions (4).

In comparing these vehicles with the current design vehicles contained in the GB90, the following points are offered:

- The Canadian tractor-semitrailer has dimensions that are similar to the GB90 Interstate semitrailers WB-62 and WB-67.
- The Canadian doubles are somewhat larger than the GB90 double (WB-60). For example, the WB-60 has a tractor wheelbase of 9.7 ft, whereas the Canadian tractor may have a wheelbase as long as 20.3 ft. Also, the trailer wheelbases of the Canadian doubles are longer than those of the WB-60. Longer wheelbases increase offtracking, which is an important factor in at-grade intersection design.

The discussion of the Canadian vehicles is presented as a guide for those involved in the design of at-grade intersections where these vehicles would be expected to operate.

OFFTRACKING

Offtracking is the phenomenon by which the rear wheels of a vehicle do not follow the same path as the front wheels. Offtracking occurs in all types of vehicles but is relatively small in passenger cars. However, offtracking by large trucks is an important consideration in intersection design.

Figure 2 shows offtracking by a truck making a turning maneuver. The most appropriate descriptor of offtracking for use in intersection design is the "swept path width," shown in Figure 2 as the difference in paths between the outside front tractor tire and the inside rear trailer tire.

Many sources, including the GB90, have identified two distinct types of offtracking. Low-speed offtracking is a purely geometrical phenomenon wherein the rear axles of a truck track toward the inside of a horizontal curve relative to the front axle. Figure 2 shows low-speed offtracking. There has been considerable research on low-speed offtracking as it pertains to level roadway pavement surfaces. Low-speed offtracking has been generally understood to be a function of the truck axle and hitch point spacings, the turn radius, and the turn angle.

Recent research by Glauz and Harwood has shown that low-speed offtracking is also influenced by pavement cross-slope, including superelevation on horizontal curves (*7*). Offtracking increases with increasing pavement cross-slope. For example, the low-speed offtracking of a truck with a 48-ft semitrailer on a curve with a 500-ft radius is increased by 20 percent on a curve with a superelevation of 0.08 ft/ft.

High-speed offtracking is a dynamic, speed-dependent phenomenon. It is caused by the tendency of the rear of the vehicle to move outward because of the lateral acceleration of the vehicle as it negotiates a horizontal curve at high speeds. High-speed offtracking is a function not only of truck and roadway geometrics but also of the vehicle speeds and the suspension, tire, and loading characteristics of the vehicle.

Intersection design can generally be based on low-speed offtracking without considering speed-dependent or superelevation effects. This is the case because most intersection turning maneuvers by trucks are conducted at low speeds on pavements with very little cross-slope.

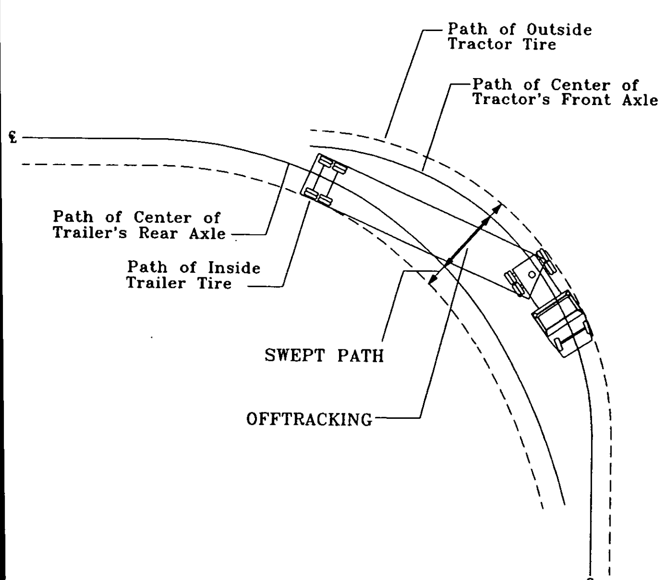


FIGURE 2 Swept path width and offtracking of a truck negotiating a 90-degree intersection turn (*2*).

Determination of Low-Speed Offtracking for Use in Intersection Design

There are four methods for determining offtracking for use in intersection design: offtracking plots and templates, computer models, offtracking charts, and offtracking equations. Each of these approaches is briefly described in the following subsections.

Turning Plots and Templates

Offtracking for use in intersection design is determined most commonly from plots depicting the turning paths of trucks. A turning angle of 90 degrees is appropriate for most intersection situations. Of course, unusual geometric situations requiring other turning angles, including skewed intersections, are not uncommon. Turning plots for various combination trucks can be found in the GB90 (*1*).

Turning templates have historically been developed using graphical techniques, such as the Tractrix Integrator (*8*). This approach has been used to create templates that can be used by designers to trace the swept path width of a vehicle to scale on the plan view of an intersection so that the geometrics of the intersection and turning roadways can be selected to accommodate a particular design vehicle (*9*).

Computer Models

A number of state departments of transportation, universities, and consultants have computer models, both PC based and mainframe, to generate truck offtracking values. These models vary in sophistication and output. For example, some models only provide turning plots of the vehicle swept paths, whereas others provide both the plots and numerical values.

The California Department of Transportation (Caltrans) has recently enhanced an existing PC offtracking model to include numerical output of offtracking and swept paths as well as the turning plot (*10*).

Offtracking Charts

With the availability of computer models with numerical output, offtracking charts can be prepared to conveniently summarize the offtracking performance of specific vehicles. Figure 3 shows the offtracking performance of a combination truck with a 48-ft semitrailer. Charts of this type for a variety of vehicles are presented by Harwood et al. (*2*), and four typical charts are presented by Glauz and Harwood (*7*). Figure 3 shows that, for a given truck, offtracking decreases with increasing turn radius. For a given radius, offtracking increases with increasing turn angle until a maximum or steady-state value is reached.

The swept path width can be determined by adding 7.58 ft to the offtracking values in Figure 3. The value of 7.58 ft is equivalent to one-half of the typical rear tractor steering axle width (6.66 ft) plus one-half of the typical rear trailer axle width (8.50 ft).

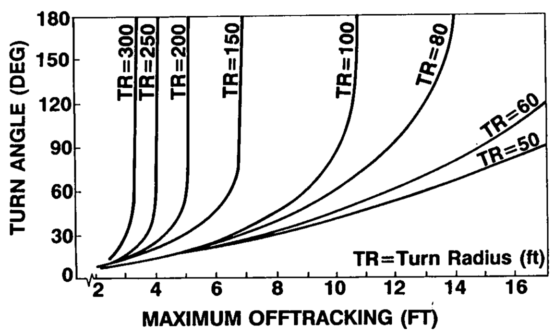


FIGURE 3 Offtracking plot for STAA single 48-ft (14.6-m) semitrailer truck with conventional tractor (*2*).

Offtracking Equations

Several published equations can be used to determine truck offtracking. All of these equations predict the maximum or steady-state offtracking (i.e., the offtracking value of the vertical portion of each curve in Figure 3). For smaller turn angles, the actual offtracking may be less than this maximum.

The best known offtracking equation is the Western Highway Institute (WHI) offtracking formula (*11*). The WHI formula is a special case of a more general formula developed by Glauz and Harwood, which includes the speed-dependent and superelevation effects (*7*).

Comparison of Truck Offtracking Performance

Table 3 summarizes the maximum offtracking of a variety of truck types of turns on 50-, 100-, and 300-ft radii and for turn angles of 60, 90, and 120 degrees. The truck axle spacings used to derive these values are similar, but not identical, to the design vehicles recommended in the GB90 (*1,2*). The table shows clearly that offtracking (*a*) decreases with increases in turn radius, (*b*) increases with increases in turn angle (until a maximum is reached), and (*c*) increases with increases in trailer length (because of increased spacing between the front and rear trailer axles). Double-trailer trucks with 28-ft trailers generally have offtracking that is less than, or equal to, even the shortest single-trailer combination trucks.

INTERSECTION SIGHT DISTANCE

AASHTO Procedures

AASHTO policy provides sight distance values for vehicles stopped on the minor approach of an at-grade intersection (i.e., Case III). Case IIIA represents the sight distance needed to complete a crossing maneuver for a stopped vehicle. Case

TABLE 3 Offtracking for Selected Combinations of Turn Radius and Turn Angle (*2*)

	Maximum offtracking (ft)[a]								
Turn radius (ft):	50			100			300		
Turn angle:	60°	90°	120°	60°	90°	120°	60°	90°	120°
Design Vehicle									
Single with 37-ft trailer (WB-50)	9.3	11.8	13.3	6.0	6.5	6.6	2.1	2.1	2.1
Single with 45-ft trailer	12.1	15.5	NA	8.0	9.0	9.4	2.9	2.9	2.9
STAA single with 48-ft trailer and conventional tractor	13.0	16.9	NA	8.8	10.0	10.5	3.3	3.3	3.3
STAA single with 48-ft trailer and long tractor	13.4	17.4	NA	9.1	10.4	10.8	3.4	3.4	3.4
Long single with 53-ft trailer	14.4	19.5	23.4	10.3	12.1	12.8	4.1	4.1	4.1
STAA double with cab-over-engine tractor	9.2	11.3	12.6	5.8	6.1	6.2	1.9	1.9	1.9
STAA double with cab-behind-engine tractor	9.6	11.9	13.4	6.0	6.4	6.4	2.1	2.1	2.1

[a]Add 7.58 ft to entries in this table to get maximum swept path width.
NA = Not available

IIIB, Curve B-1 represents the sight distance to the left for a left-turning vehicle. Cases IIIB and IIIC (turning left or right onto a cross road), Curves B-2a & Ca and B-2b & Cb represent the sight distances to a major-road vehicle in the lane being entered when the major-road vehicle is traveling at design speed or reducing to average running speed, respectively.

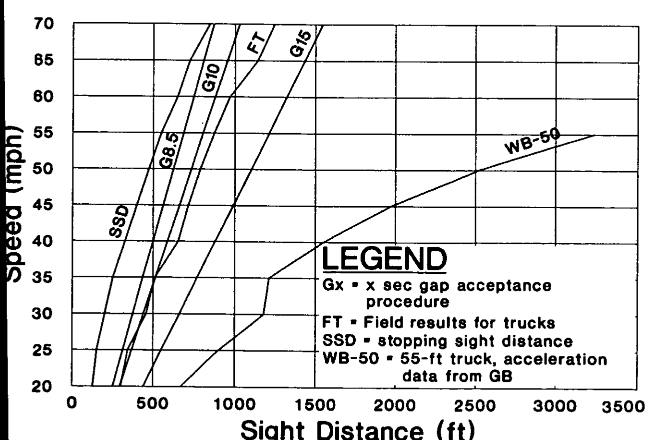
The intersection sight distance (ISD) procedures presented in GB84 and GB90 are based primarily on consideration of the passenger car as the design vehicle. However, highway design and operational criteria should consider the characteristics of all vehicles using a facility with reasonable frequency. AASHTO policy provides actual sight distance values for trucks in Case IIIA only.

The intersection sight distance discussions presented in the GB84 for Cases IIIB and IIIC lacked sufficient information to derive the AASHTO sight distance values (shown in GB84, Figure IX-27). Equations were developed to reproduce the GB84 ISD values on the basis of specific information presented in the GB84 (*2,5*). Using truck data information from the GB84 and the developed equations, WB-50 sight distances were determined for the different Case IIIB and IIIC sight distance procedures. The sight distance values for a turning WB-50 when the major-road vehicle reduces speed to the average running speed are shown as curve WB-50 in Figure 4.

Operational experiences indicate that sight distances as long as 3,000 ft (see Figure 4) are not necessary for safe operations at intersections even when trucks are present. Very few intersections have such long sight distances available, and it is unlikely that drivers could accurately judge the location and speed of an oncoming vehicle at a distance of 3,000 ft. Rather, the results indicate that using the procedures that are based on the GB84 for Cases IIIB and IIIC (for truck intersection sight distance determinations) are not practical. Further investigation into the operations at intersections is needed to determine viable sight distance values for trucks.

Field Observations

Intersection sight distance values were estimated using the findings from pilot field studies as input into the developed



IGURE 4 Comparison of five-axle truck sight distances for ight and left turns.

ISD equations (*2*, paper by Fitzpatrick et al. in this Record). Data were collected for the following parameters: minor-road vehicle acceleration, major-road vehicle deceleration and speed reduction, and the minimum separation between the major-road vehicle and the turning vehicle.

The FT curve in Figure 4 shows the sight distance values for five-axle trucks. Although the values are based on limited pilot field study, the findings illustrate the difference in required sight distance values when data from actual intersection operations are considered. The five-axle truck results are between 55 and 73 percent less than the values obtained from using the Green Book truck data in the developed equations.

Gap acceptance information was also collected in the referenced field studies. These data were used to develop an alternative method for determining intersection sight distance. Figure 4 also shows the sight distances based on selected accepted gap lengths of 8.5, 10.0, and 15.0 sec. These gaps represent the following:

- 8.5 sec, the 50 percent probability of a five-axle truck driver accepting a gap;
- 10.0 sec, the 85 percent probability of a five-axle truck driver accepting a gap at a high-volume (20,000 major road ADT) location; and
- 15.0 sec, the 85 percent probability of a five-axle truck driver accepting a gap at a low-volume (7,000 and 14,000 major road ADT) location.

The results from the pilot field studies indicate that the sight distances used by drivers in an operational setting are significantly less than the sight distances calculated using GB84 truck characteristics. Alternative procedures for determining sight distances that also consider drivers' visual limitations need to be considered. Both the field studies and the sensitivity analyses also indicate the need to update truck acceleration data. A complete discussion of the field studies and intersection sight distance is contained elsewhere (*2*, paper by Fitzpatrick et al. in this Record).

Ramp Terminals

Sight distance criteria for ramp terminals are intended to ensure that a vehicle stopped at the terminal will have adequate time to turn left and clear the intersection without colliding with a vehicle coming from the left. Ramp terminals should be designed on the basis of the same sight distance design elements as those used for other at-grade intersections. Therefore, sight distance criteria for ramp terminals are similar to the Curve B-1 procedure. An added sight distance consideration is the location of bridge parapet walls or bridge railings.

CHANNELIZATION

Channelization serves to control and direct traffic movement. The GB90, Table IX-4, provides minimum design dimensions for various oblique-angle turns. As indicated earlier, the turning characteristics of large trucks, such as offtracking and swept path width, require special consideration in the geometric layout of at-grade intersections. If the curb radius is large enough that trucks can make right turns without encroaching on adjacent lanes, the paved area at the intersection can become so large that through drivers may not understand where to position their vehicles. In such instances, it becomes necessary to construct a channelizing island to properly control traffic. If the curb radius is so small that trucks cannot make right turns without encroaching on adjacent lanes, the truck either encroaches and interferes with adjacent traffic or it does not encroach and its rear wheels run over and possibly damage the curb or shoulder. In addition, the truck's front overhang may strike traffic control devices located near the outside of its turning path, or the trailer's right rear tire may strike those devices located near the inside of its turning path when offtracking.

A study of at-grade, right-turn maneuvers by large trucks established channelization guidelines in a form similar to the GB90, Table IX-4, information (*12*). The cross street and swept path widths were identified on the basis of a computer simulation program, Truck Offtracking Model (TOM), developed by Caltrans (*10*). The design vehicles selected were two singles (WB-50 and WB-55), two doubles (WB-70 and WB-105), and one triple (WB-100). Truck dimensions were based on the then-current GB84 and supporting literature. The offtracking characteristics of the WB-55 and WB-70 were very similar to the STAA 48-ft single trailer and STAA doubles, respectively.

In addition to the design vehicles, parameters considered in the Texas study were curb return simple radius and degree of turn angle (*12*). The minimum turning radii of the outside and inside wheelpaths for each of five design vehicles varied slightly from AASHTO policy because of shorter tractor and longer trailer axles spacing assumptions. The minimum turning radii represented turns negotiated at less than 10 mi/hr. This assumption minimizes the effects of driver characteristics and the slip angle of the wheels.

The Texas study found that as the curb return radius increases toward 200 ft, the area of an island becomes larger and the width of a turning lane decreases (*12*). The size of islands for large turning angles indicates that the size of the otherwise unused and uncontrolled areas of pavement can be eliminated by the use of channelization. Turning roadways for flat-angle turns, less than 75 degrees, involve relatively large radii and require channelization designs to fit site controls and traffic conditions. Furthermore, since truck configurations follow a spiral path into a curve, it would be desirable to fit the edge of the pavement closely to the minimum path of the design vehicle by using three-centered compound curves or simple curves with tapers to minimize the amount of unused pavement.

TRAFFIC ENGINEERING ELEMENTS

Passenger Car Equivalencies

Molina et al. developed passenger car equivalencies (PCEs) for trucks traveling straight through a level, signalized intersection on the basis of vehicle type and position (*13*). PCE values were determined for single-unit (SU) trucks and four-axle and five-axle truck tractor-semitrailer combinations. The recommendations of the study indicated that truck types should be distinguished to account for the presence of heavy versus light trucks when analyzing the capacity of an at-grade intersection.

Although the 1985 HCM accounts for trucks as part of the heavy-vehicle adjustment procedure, the heavy-vehicle factor was found to be different for SU trucks and tractor-semitrailer truck combinations (*19*). The authors, using the heavy-vehicle adjustment values found in Table 9-6 of the HCM, calculated a PCE value of 1.5, which they indicate probably is the average of the PCE values for all heavy truck types at signalized intersections. PCE values of 3.7 and 1.7 are recommended in their study as being representative of heavy and light trucks, respectively.

Vehicle Change Interval

Harwood conducted a sensitivity analysis of the differences in vehicle change interval requirements for passenger cars and trucks (*2*). In his analysis, Glauz compared the vehicle change interval requirements based on the FHWA *Traffic Control Devices Handbook* (TCDH) criteria for passenger cars and from information in a 1984 FHWA study (*15,16*). The criteria that were varied in the sensitivity analysis included perception-reaction time, deceleration rate, percent roadway gradient, length of vehicle, and width of intersection.

Since no data were available for perception-reaction time requirements for trucks, the perception-reaction times for trucks

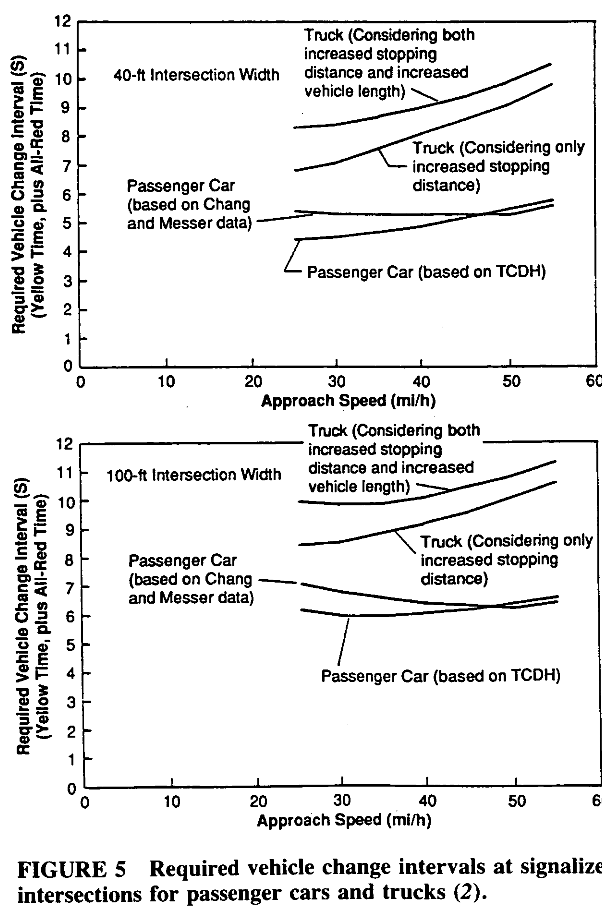


FIGURE 5 Required vehicle change intervals at signalized intersections for passenger cars and trucks (*2*).

TABLE 4 Additional Pavement Construction Costs To Accommodate Design Vehicles Larger Than AASHTO WB-50 Truck at Urban Intersections (*2*)

Posted or 85th percentile speed (mi/h)	Distance from warning sign to potential hazard (ft)[a]						
	Condition A[b] (high judgment needed)	Condition B[c] (stop required)	Condition C[d] (deceleration to stated advisory speed [mi/h])				
			10	20	30	40	50
20	250	e	e	NA	NA	NA	NA
25	325	e	125	e	NA	NA	NA
30	425	175	225	150	NA	NA	NA
35	500	250	325	250	100	NA	NA
40	600	325	450	375	225	NA	NA
45	675	425	600	500	325	175	NA
50	775	525	750	650	525	325	NA
55	850	650	900	825	675	500	225
60	950	775	1,075	1,000	875	675	425
65	1,025	900	1,225	1,200	1,050	850	600

a All distances are based on the assumption that the warning sign is legible to drivers for 125 ft (38 m) in advance of the sign. For large [48-in by 48-in (122-cm by 122-cm)] signs, the legibility distance can be increased by 200 ft (61 m) and each of the entries in this table can therefore be reduced by 75 ft (23 m).
b Includes 12.0-s Perception-Intellection-Emotion-Volition (PIEV) time.
c Includes 2.5-s PIEV time and deceleration rates for driver with 70% braking control efficiency driver.
d Based on comfortable deceleration rate equal to two-thirds of the deceleration rate used for Condition B.
e No suggested minimum distance provided; at these speeds, sign location depends on physical conditions at site.
NA = Not available

approaching yellow signal indications were assumed to be equal to the values observed in the Chang and Messer study (*16*). The deceleration rate was assumed to be 5 ft/sec^2, which is a comfortable rate on a dry pavement but may be a critical rate for some drivers on a poor, wet pavement surface. Assumed length of vehicle values were 19 ft for passenger cars and 75 ft for trucks. Width of intersection used for comparison included a 40-ft-wide intersection as a "moderate" value and 100 ft for a "wide" intersection. Figure 5 compares the required vehicle change interval for trucks based solely on their increased braking distances with the required vehicle change interval incorporating both their increased braking distances and their increased lengths.

The analysis demonstrated that trucks may require vehicle change intervals that are 40 to 110 percent longer than passenger cars, depending on approach speed, approach grade, and intersection width. Because longer change intervals would increase delays on the other approaches, additional study of the overall operational and safety impacts is necessary before any modification is recommended in existing criteria.

Sign Placement

MUTCD Section 2C-3 presents criteria for advance placement of warning signs to provide adequate time for drivers to perceive a potentially hazardous condition, identify the condition, decide what maneuver to make, and begin to perform that maneuver (*17*). Current criteria for horizontal and vertical placement of signs are not based on any explicit vehicle characteristic. However, the criteria for longitudinal placement of warning signs depend significantly on the deceleration capabilities of a vehicle.

Harwood et al. have developed a modified version of the MUTCD criteria for the placement of advance warning signs on the basis of their findings regarding truck stopping distances and comfortable deceleration rates. Table 4 presents the modified criteria for trucks (*2*). However, there are no available data on whether trucks encounter any safety problems at signs placed in accordance with existing criteria or whether there would be any safety benefits from adopting the modified criteria. The recommended advance warning sign distances for trucks could be reduced if antilock brakes were to become widely used.

CONCLUSIONS

This paper has presented several key design considerations to accommodate large trucks at intersections. The key features of the principle elements are discussed in the following subsections.

Physical Characteristics

Although the GB90 currently includes 15 design vehicles, future truck combinations will probably follow some form of the Turner-type truck. Information provided regarding Canadian trucks indicates that the Canadian tractor-semitrailer combinations are similar to the AASHTO WB-62 and WB-67 types: Canadian doubles are generally larger than the AASHTO GB90 WB-60 double-trailer combinations.

Offtracking

Turning roadway design speed governs the type and amount of offtracking that the truck-trailer units will generate. Fundamentally, low-speed offtracking decreases with increased turn radius, increases with increased turn angle, and increases with trailer length. Double (twin) trailers of 28-ft length typically offtrack less than single-trailer combination trucks.

Intersection Sight Distance

Recommended sight distances for truck turning maneuvers at intersections are not explicitly discussed in the GB90. Equations have been developed to reproduce the GB84 and GB90 procedures. Field observations of actual truck turning maneuvers yield shorter intersection sight distance values than calculated using the GB84 and GB90 procedures and reported truck acceleration information.

Channelization

Specific guidelines are not available to fully evaluate the need and scope of intersection channelization to accommodate trucks on turning roadways. It seems desirable to fit the pavement edges of the turning roadway to a spiral or taper geometry to minimize excess pavement and to conform more closely to the truck's offtracking.

Traffic Engineering Elements

Several studies have investigated the effects of large trucks at signalized intersections. Preliminary PCE values of 1.7 and 3.7 have been proposed for SU and tractor-semitrailer truck combinations, respectively. A sensitivity analysis has demonstrated that trucks require change intervals that are significantly longer than passenger cars. Additional study is necessary to fully evaluate the impacts of modifying current TCDH criteria.

A revised tabulation of the MUTCD criteria for the advance placement of warning signs has been developed. The modifications were based on increased stopping distances and comfortable deceleration rates for trucks.

ACKNOWLEDGMENT

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REFERENCES

1. *A Policy on Geometric Design of Highways and Streets.* American Association of State Highway and Transportation Officials, Washington, D.C., 1990.
2. D. W. Harwood, J. M. Mason, W. D. Glauz, B. T. Kulakowski, and K. Fitzpatrick. *Truck Characteristics for Use in Highway Design and Operation.* Reports FHWA-RD-89-226 and FHWA-RD-89-227. FHWA, U.S. Department of Transportation, 1990.
3. *Special Report 227: New Trucks for Greater Productivity and Less Road Wear—An Evaluation of the Turner Proposal.* TRB, National Research Council, Washington, D.C., 1990.
4. *Recommended Regulatory Principles for Interprovincial Heavy Vehicle Weights and Dimensions.* Implementation Planning Subcommittee of the Joint RTAC/CCMTA Committee on Heavy Vehicle Weights and Dimension, Sept. 1987.
5. K. Fitzpatrick and J. M. Mason. Review of AASHTO Case III Procedures for Intersection Sight Distance. In *Transportation Research Record 1280*, TRB, National Research Council, Washington, D.C., 1990, pp. 216–226.
6. *Special Report 211: Twin Trailer Trucks—Effects on Highways and Highway Safety.* TRB, National Reserach Council, Washington, D.C., 1986.
7. W. D. Glauz and D. W. Harwood. Superelevation and Body Roll Effects on Offtracking of Large Trucks. In *Transportation Research Record 1303*, TRB, National Research Council, Washington, D.C., 1991, pp. 1–10.
8. D. S. Millar and C. M. Walton. Offtracking of the Larger Combination Commercial Vehicles. In *Transportation Research Record 1026*, TRB, National Research Council, Washington, D.C., 1985, pp. 62–65.
9. *Turning Vehicles Templates: A Transportation Design Aid.* Jack E. Leisch and Associates, Evanston, Ill., 1977.
10. *Truck Offtracking Model (TOM), Program Documentation and Users Guide.* Division of Transportation Planning, California Department of Transportation, Sacramento, 1985.
11. *Offtracking Characteristics of Trucks and Truck Combinations.* Research Committee Report 3. Western Highway Institute, San Francisco, Calif., 1970.
12. N. Straub, D. B. Fambro, and J. M. Mason, Jr. *Channelization Guidelines to Accommodate Longer and Wider Trucks at At-Grade Intersections.* Report FHWA/TX-87/397-3. FHWA, U.S. Department of Transportation, 1987.
13. C. J. Molina, C. J. Messer, and D. B. Fambro. *Passenger Car Equivalencies for Large Trucks at Signalized Intersections.* Report FHWA/TX-87/397-2. FHWA, U.S. Department of Transportation, 1987.
14. *Special Report 209: Highway Capacity Manual.* TRB, National Research Council, Washington, D.C., 1985.
15. *Traffic Control Devices Handbook.* Federal Highway Administration, U.S. Department of Transportation, 1983.
16. M. S. Chang and C. J. Messer. *Engineering Factors Affecting Traffic Signal Yellow Time.* Report FHWA/RD-85/054. FHWA, U.S. Department of Transportation, 1984.
17. *Manual on Uniform Traffic Control Devices.* Federal Highway Administration, U.S. Department of Transportation, 1978.

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Safety Improvements at Intersections on Rural Expressways: A Survey of State Departments of Transportation

JAMES A. BONNESON, PATRICK T. MCCOY, AND JESS E. TRUBY, JR.

The current state of the practice of measures used to improve traffic safety at intersections on rural expressways is described. The description is based on the results of a recent survey of 49 state highway departments. In general, highway departments use their access control policy and a variety of safety improvement measures at locations with poor safety records to minimize accident potential. The access control policy typically specifies the justification for and frequency of access openings and median openings. Most states indicated that one access opening is provided per abutting parcel that cannot be served by other means. In contrast, median openings are typically provided only at intersections of the expressway and other public roads. Safety improvement measures identified by the survey respondents were categorized as either traffic control measures or geometric design measures. Seventy-four percent of the states indicated that they consider traffic signal control and flashing beacons for application at high-accident locations. Thirty percent of the states consider turn lane additions or modifications at high-accident locations. One modification of expressway left-turn lane design that appears to have particular merit is the offset left-turn bay. In this design, opposing left-turn bays on the expressway are laterally offset such that stopped vehicles in the bay do not block the sight lines of opposing left-turn vehicles.

A rural expressway can be functionally classified as a minor arterial that in most situations is designed as a four-lane divided facility. A rural expressway is commonly used as a high-speed linkage between cities and larger towns and as a bypass around urban areas.

Rural expressways can be characterized as high-speed facilities having partial control of access. Access to a rural expressway is usually limited to intersections with all public roadways, provided that a minimum spacing can be maintained. Access to adjacent properties from the expressway is provided only when access by alternative routes cannot be attained. A typical at-grade intersection on a rural expressway is shown in Figure 1.

The combination of high-speed operation and only partial access control can adversely affect the safety of rural expressways. Contributing factors to accidents at intersections on rural expressways commonly include the following:

- Some turning and crossing drivers are unable to judge the speed and distance (i.e., arrival time) of approaching expressway drivers.
- Some crossing drivers are unable to judge the lengthy crossing time required to clear an at-grade intersection.
- Some left-turn drivers are unable to see oncoming expressway drivers because the median design is such that opposing left-turn vehicles block one another's line of sight.
- An at-grade intersection may be inconsistent with the expressway driver's expectancy. Because of partial access control, at-grade intersections on expressways are infrequently encountered and thus generally unanticipated by expressway drivers. Moreover, typical driver expectancy for high-speed roadways is that they have full access control like that found on the more frequently occurring freeways and Interstate highways.
- High deceleration rates are sometimes required of a stopping expressway vehicle.
- The speed differential between expressway through traffic and traffic entering or exiting the expressway may be large.

To offset the adverse impact of these factors on safety, state highway departments incorporate both proactive and reactive measures. One of the most important proactive measures is the access control policy because it preserves the quality of traffic service by regulating the frequency and location of all access to the expressway. Reactive measures usually relate to various safety improvement measures, which can generally be categorized as either traffic control measures or geometric design measures. Traffic control measures may include signalization, delineation, and signing. Geometric design measures commonly include auxiliary lanes, channelization, and grade separation.

SURVEY OF STATE HIGHWAY DEPARTMENTS

The Nebraska Department of Roads is currently examining the design and operation of at-grade intersections on rural expressways. Accident histories at several of these intersections suggest that safety problems exist stemming from the combination of high-speed operation and partial access control policies. Potential corrective measures include offsetting the expressway left-turn lanes, signalization, and conversion to an interchange design.

To weigh the relative merit and cost-effectiveness of these and other potential corrective measures, a review of the literature was conducted. The review indicated that there is relatively little published about traffic safety at intersections

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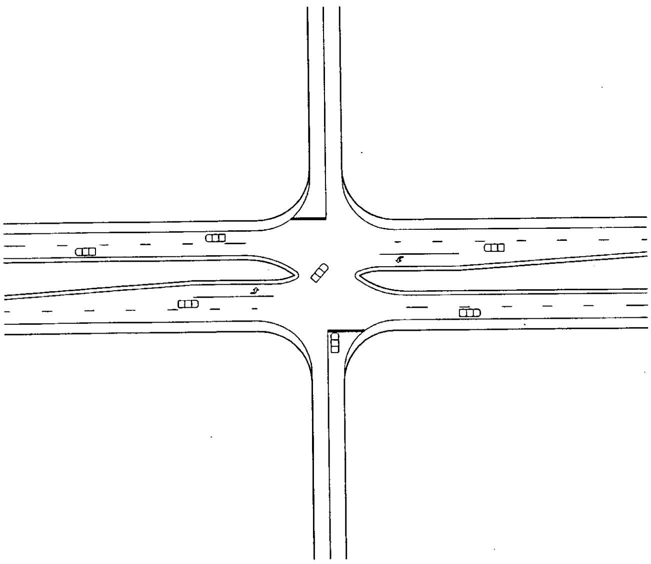

FIGURE 1 Typical unsignalized intersection on a rural expressway.

on rural expressways. Similarly, little published material was found that addressed the relative merits of alternative left-turn bay designs.

Although the literature review did not yield anything that was directly applicable to safety problems at rural expressway intersections, a considerable amount of information was found that dealt with areas related to this subject. For example, the literature review indicated that there was a large body of research examining traffic accidents at signalized intersections and at interchanges on the Interstate system. In the event that more directly relevant research is not found, it is anticipated that some of the findings and conclusions from this research could be extended to intersections on rural expressways.

The literature was also consulted for research on warrants or guidelines for the installation of traffic signals and for the construction of an interchange. In this regard, the U.S. Department of Transportation's *Manual on Uniform Traffic Control Devices* (MUTCD) (*1*) provides warranting criteria for the signalization of rural intersections. These warrants provide some guidance regarding the need for traffic signals based on traffic volume and accident history. In contrast, there are no nationally recognized warranting criteria for interchanges.

Concurrent with the literature review, a questionnaire was sent to highway departments in 49 states. Questions pertained to the state's current rural expressway design practice. Specific questions were asked about the access control policy, traffic control measures, and geometric design considerations used by each state.

The response to the survey questionnaire was quite good with 42 (86 percent) of the 49 states responding. On the basis of these responses, it was found that 30 states (71 percent) are building (or have built) rural expressways with at-grade intersections. In addition to completing the questionnaire, most of these 30 states sent additional information detailing particular design or policy issues that were relevant to rural expressways.

ACCESS CONTROL POLICY

One question included in the survey inquired about the state's access control policy for rural expressways. Twenty-eight states responded to this question; 9 of the 28 sent copies of their access control policy. In general, most of the 28 states have formal policies for rural expressways that provide for some form of partial access control. A few states indicated that they apply their Interstate highway access control policy to their rural expressways, which implies full access control.

In general, access to the expressway is regulated by the permit of access openings, median openings, and interchanges. In most cases, responses to the survey referred to one or more of these access types. Each of these access types and a summary of the comments made about them will be discussed in the following subsections.

Access Openings

Access openings refer to a point of direct access to the through lanes of the expressway. An access opening will always provide for right turns into and out of the property. However, if a median opening is provided opposite to the access opening, the access will also include left turns into and out of the property.

Almost all of the states that responded indicated that one access is provided per parcel that cannot be served economically by another means (e.g., frontage road, service road, and parallel street). Some states indicated that access is not provided to commercial properties in new construction or major reconstruction. However, these few states also indicate that commercial access will be permitted on existing rura expressways or on those under minor reconstruction.

Several states have adopted minimum, desirable, and/o maximum spacings of access openings to minimize the inter-ruption to the through traffic flow. Typical minimum spacing range from 440 to 880 ft. Desirable and maximum spacing recommended by one state are 1,320 and 1,760 ft, respec-tively. Another state specified that access openings should no be located within 300 ft of other median openings unless th access opening is coincident with the median opening.

Median Openings

Median openings (or crossovers) refer to locations along th expressway where traffic can legally access or cross over t the far-side through lanes. Median openings are generall provided at at-grade intersections; however, they are als provided at various locations along the expressway to facilitat U-turn movements.

Responses varied as to the provision of median opening In general, most states indicated that median openings we provided only at public roads (i.e., county, state, and feder highways) and subject to a minimum spacing requiremen Median openings are not typically provided for adjacent pro erties (i.e., residential, agricultural, or commercial), althoug a few states recognized the possibility of special situatio wherein a median opening would be acceptable (e.g., existi opening, no median crossing for ½ mi in either directio

One state indicated that a median opening was acceptable when the commercial property generated relatively high traffic demands (i.e., 350 left-turn vehicles per day).

Most states specified minimum, desirable, and/or maximum spacing requirements for median openings. Minimum requirements ranged from 330 to 2,640 ft. The most frequent minimum spacing was about 1,300 ft. Desirable spacings ranged from 2,500 to 5,000 ft. Only one state recommended a maximum spacing for median openings, which it specified as 1.0 mi.

SAFETY IMPROVEMENT MEASURES

One question on the survey asked for information about the types of corrective measures that the states apply (or would apply) to high-accident unsignalized at-grade intersections on rural expressways. The response to this question was varied. In general, treatments considered range from low-cost solutions such as signing to high-cost solutions such as grade separation. The survey responses are summarized in Table 1.

As Table 1 indicates, the most frequently cited corrective measure was signalization (which includes both traffic signal control and flashing beacons), followed by signing improvements. Corrective measures relating to geometric design elements were also cited frequently; however, within this category, the specific improvement type ranged widely from left-turn bays to rumble strips. An examination of the measures given in Table 1 suggests that almost all are intended to increase the likelihood of attracting the expressway driver's attention, to attract the driver's attention further in advance of the intersection, and to provide more restrictive traffic regulation through the intersection.

Traffic Control Measures

In general, the selection of appropriate corrective measures is based on the type of accidents occurring at each location. The accidents are the result of a wide variety of factors that are individually insignificant but in combination create unsafe situations, which can be causally related to an increase in accidents. Because of the wide variety of accident contributing factors and the variability in their overall impact, there are many potential accident reduction treatments.

The questionnaire asked the state highway departments to list the various corrective measures that they had used at high-accident at-grade intersections on rural expressways. Responses varied widely; however, the more commonly used techniques could be categorized as traffic control measures. Measures requiring geometric design improvements were cited much less frequently.

In general, the responding states indicated that traffic signals (i.e., traffic control signals, traffic beacons) were the most commonly applied corrective measures. Other traffic control measures that were mentioned included specialized or enhanced signing and marking applications. The following paragraphs elaborate on the frequency and extent to which traffic control devices are used as a corrective measure.

Signalization

In general, two types of signalization are considered by the states as corrective measures at at-grade intersections on rural expressways: traffic signal control and flashing beacons. Traffic signal control refers to the regulation of traffic by means of

TABLE 1 Summary of Corrective Measures at Unsignalized Intersections

Corrective Measures	Number of States			Percent[1] of States		
Signalization	17			74		
Traffic signal control		12			52	
Flashing beacon		11			48	
Intersection control beacon			9			39
Stop sign beacon			2			9
Hazard identification beacon			2			9
Signing Improvements	8			35		
Advance signing		6			26	
Increase sign size		2			9	
Reduce sign clutter		1			4	
Exclusive Lanes for Turning Traffic[2]	7			30		
Grade Separation/Interchange	5			22		
Reduce Speed Limit	4			17		
Partial Lighting	3			13		
Rumble Strips	2			9		

[1]Frequencies based on responses from 23 states.

[2]Treatments mentioned include: add right-turn bay, lengthen left-turn bay, add median acceleration lane, add right-turn acceleration lane, offset left-turn lanes, and prohibit turns by closing median.

a single signal controller regulating signal heads for each of the entering movements. Flashing beacons include those used to draw attention to warning signs and those used to control the intersection. Seventy-four percent of the states indicated that they consider one (or both) of the two types of signalization at high-accident locations. Each measure was listed by about one-half of the responding states as a corrective measure used at locations with high accident rates.

When asked whether other criteria were used (in addition to accidents) to justify traffic signalization, 30 percent of the responding states indicated that their decision was based on traffic volume only. In contrast, 52 percent of the states use accident rates or a combination of accident rates and traffic volume in determining the need for traffic signal control. Thirty percent of the states responding indicated that they did not signalize at-grade intersections on rural expressways or that they did so only when alternative measures had been considered first.

The most commonly cited criteria used to determine the need for signalization were the state's traffic signal warrants. In two instances, the states indicated that they used the signal warrants specified in the MUTCD (*1*). Warrants of this type include both traffic demand and traffic accidents as the warranting criteria. A few states were not specific as to the criteria used but indicated that both accidents and traffic volume were considered when determining the need for traffic signal control.

Other criteria were also used by some states to determine the need for traffic signal control. One state indicated that the installation of a traffic signal was justified at all intersections of a rural expressway with other marked routes. Another state indicated that all major intersections on six-lane expressways were signalized.

Median width was another criterion considered in the decision to use traffic signal control. Several states indicated that they consider signalization only at intersections with "narrow" medians. This restriction relates to the inefficient and unsafe nature of an intersection with a wide median. Specifically, a wide median can lead to longer lost times between signal phases and a larger area of uncontrolled pavement. One state defined a narrow median as being less than 20 ft wide. Another state suggested that medians more than 50 ft wide were too wide for efficient signalization.

Flashing beacons are used by many states as a corrective measure. Flashing beacons include intersection control beacons, Stop sign beacons, and hazard identification beacons. An intersection control beacon is suspended over the intersection and has flashing yellow and red indications for the major and minor approaches, respectively. A Stop sign is typically located on each minor approach in conjunction with this beacon. Thirty-nine percent of the responding states use intersection control beacons in situations where high accident rates indicate a hazardous location.

A Stop sign beacon is suspended over the intersection and has flashing red indications for both the major and minor roadways. This type of beacon implies four-way stop control and is typically used in conjunction with Stop signs on all approaches. Nine percent of the responding states indicated that they considered this treatment at high-accident intersections on rural expressways.

Hazard identification beacons are used to supplement warning or regulatory signs. Nine percent of the responding states indicated that they have installed this type of beacon at high-accident expressway intersections.

Other Traffic Control Measures

Other traffic control measures used as accident countermeasures include signing improvements, reduced speed limits, and rumble strips. Signing improvements were considered by 35 percent of the states responding. These improvements typically include the addition of advance signing, increase in sign size, and reduction of existing sign clutter. Advance signing was cited as a corrective measure most frequently (26 percent). Increasing sign size to 48 × 48 in. was considered by 9 percent of the states, whereas reduction of existing sign clutter was mentioned in only 4 percent of the responses.

Reducing the speed limit on the expressway was considered by 17 percent of the states as a viable accident reduction measure. Nine percent of the states indicated that they had considered the use of rumble strips on the minor road approaches to the intersection.

Geometric Design Measures

Expressway Left-Turn Bay and Median Width

Left-turn bay and median designs for efficient operation ar[e] frequently in conflict with those designs for maximum safety[.] This conflict has been the subject of some controversy re[-]garding the optimal design combination. The underlying prob[-]lem is the lengthy sight distances crossing and left-turnin[g] drivers need as a result of the high-speed operation of th[e] expressway. The problem is often aggravated by the relativel[y] wide medians commonly found on rural expressways. Wid[e] medians increase the clearance distance of both crossing an[d] turning drivers and thereby increase the sight distance neede[d] to ascertain the safety of the crossing or turning maneuver.

To investigate the magnitude and extent of the left-tur[n] safety problem on rural expressways, a series of questio[ns] about median and left-turn bay design was included in th[e] survey. One question inquired about the median width use[d] between intersections. The response to this question was va[r]ied. Some states have only a minimum width criterion, othe[rs] have minimum and desirable widths, and still others hav[e] minimum and maximum widths. A summary of the respons[es] to this question is presented in Table 2.

TABLE 2 Summary of Rural Expressway Median Widths

Median Width Between Intersections	Range (feet)	Median Val[ue] (feet)
Minimum	4 - 84	46
Desirable	40 - 66	48
Maximum	40 - 104	48

As Table 2 suggests, median widths of 40 to 50 ft are most commonly used for rural expressways. Widths in this range have generally been found to provide a good balance between overall right-of-way width and safe traffic operations. Medians less than 40 ft wide may not provide adequate protection from errant vehicles, whereas medians more than 70 ft wide are probably not cost-effective with respect to added safety.

Although wide medians provide protection from encroachment by opposing traffic, they can introduce operational problems at unsignalized at-grade intersections. In the case of the left-turn maneuver off the expressway, two potential problems exist. First, a wide median combined with traditional left-turn bay design places opposing left-turn movements directly in each other's line of sight to oncoming traffic (see Figure 2). Second, the travel paths of these left-turn movements tend to be overlapped such that simultaneous movement of opposing left-turn movements can result in a head-on collision. Wide medians also increase the size of the intersection conflict area and make it difficult for crossing drivers to safely clear the intersection.

In recognition of the safety problems associated with wide medians at intersections, about one-third of the states that build rural expressways consider alternative median widths or left-turn bay designs, or both. Alternative median widths include those that are less than 20 ft (narrow) and those that are more than 100 ft (wide). The benefits of narrow median widths are reduced sight distance blockage and turn path conflict among opposing left-turn movements as well as shorter clearance path lengths. Medians of 100 ft or more also separate opposing left-turn movements and minimize clearance paths by forming two closely spaced but separate intersections.

The most common alternative left-turn treatment has left-turn bays that are laterally offset to eliminate the sight distance obstruction created by opposing left-turn movements. Figure 3 shows two methods of offsetting left-turn bays. The method shown in Figure 3a has both left-turn bays angling through the median. Experience with this configuration indicates that some obstruction to opposing left-turn driver sight distance can be incurred when the storage area contains several stopped vehicles. The method shown in Figure 3b has both left-turn bays offset and parallel to the through lanes. With this design all queued left-turn vehicles are removed from the opposing left-turn driver's line of sight.

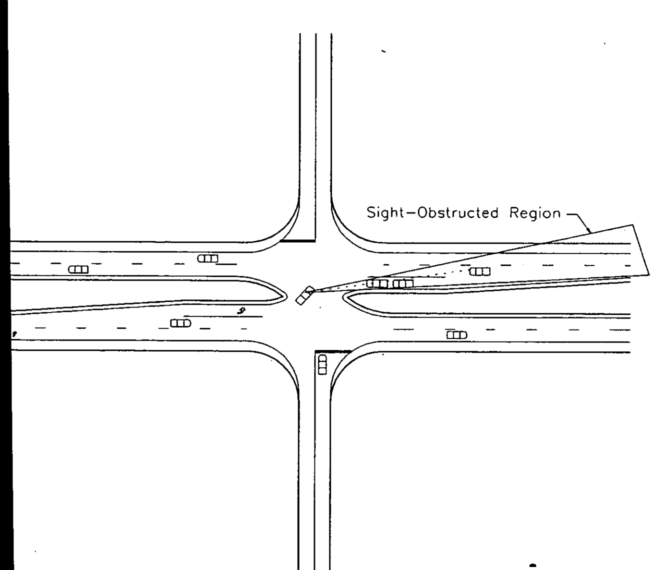

FIGURE 2 Sight obstruction to left-turning vehicles.

Responses to the survey indicate that laterally offset left-turn bays are generally considered for medians that are more than 30 ft wide. This trend stems from the fact that sight distance restrictions and turn path conflicts associated with traditional turn bay designs (as shown in Figure 2) tend to increase in severity with increasing median width. Although most states consider the offset left-turn bay design, they point out that it is not a design standard. It is most often considered where wide medians exist and left-turn accident problems have been encountered. In addition, several states indicated that this design was considered primarily for signalized intersections with permitted or protected/permitted left-turn signal phasing.

Concerns have been raised about the safety of the offset left-turn bay design. Although the design improves left-turn sight distance and lessens turn path conflicts, several states suggest that the small island on the right side of the offset turn bay may introduce some safety problems. This problem stems from the unusual nature of this design—most drivers are unaccustomed to driving in offset turn bays. There is a concern that drivers would not use the bay as intended since the small island would be flush and painted in all rural applications. If turning drivers do not respect the intended channelization, the safety benefits that this design offers may be negated.

Another, less frequently used left-turn treatment is shown in Figure 4. This treatment prohibits left turns at the intersection but permits them at one-way median U-turn lanes downstream of the intersection. This indirect left-turn arrangement requires a relatively wide median width and is used primarily as a part of stage construction where the right-of-way is ultimately used for an interchange.

Interchanges

Interchanges provide the safest access to a high-speed facility. Traffic can access the expressway through lanes via ramps that promote high-speed merge or diverge maneuvers rather than the direct entry and slower turn speeds associated with access openings or median openings. In general, 22 percent of the states responding consider interchanges to be a corrective measure for intersections with high accident rates.

Two states that responded to the survey indicated that they design all new rural expressways that bypass cities as full-access control facilities. In this regard, they construct interchanges at all major intersections along the bypass. This policy was adopted because of the high accident rates found at many existing at-grade intersections on rural bypass/expressways. Potential benefits of this design, in addition to improved safety, include the up-front dedication of right-of-way and the provision of surplus capacity for future traffic growth.

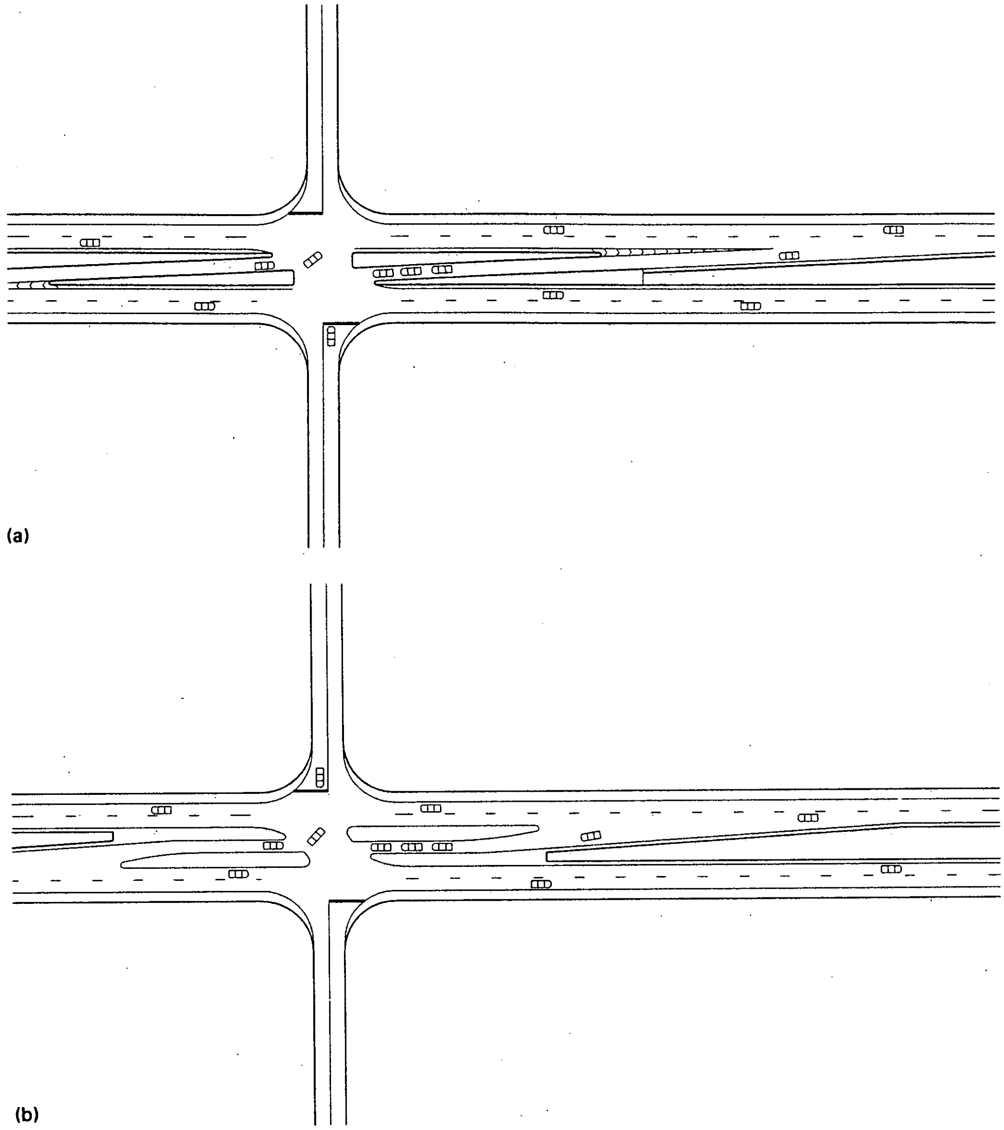

FIGURE 3 Alternative left-turn bay designs: (*a*) tapered offset left-turn bays, (*b*) parallel offset left-turn bays.

Other Geometric Design Measures

Other geometric design measures that were considered varied in application but shared the goal of increasing the separation between turning and through traffic. The impetus for this common goal stems from the higher accident potential associated with traffic streams having high speed differentials. Typical measures recommended included adding a right-turn bay, lengthening the left-turn bay, adding a median acceleration lane, and adding a right-turn acceleration lane. The fact that none of these measures was cited by more than one state reflects their less frequent application resulting from higher implementation costs.

In general, specific "warranting" criteria were not cited for the turn bay or acceleration lane improvements other than the fact that they would be considered at all locations where they could potentially reduce turning or merging accidents One state, however, indicated that its design policy for rura multilane highways recommends the use of left- and right turn bays at all public road access points when the desig speed of the highway is 40 mi/hr or more.

CONCLUSIONS

The combination of high-speed operation and only partia access control can have an adverse impact on the safety o rural expressways. To mitigate this impact, several measure are frequently used by state highway departments. One is th access control policy, which is used to regulate the frequenc and location of all access to the expressway. Measures use at at-grade intersections along the expressway include traffi control devices and geometric design features.

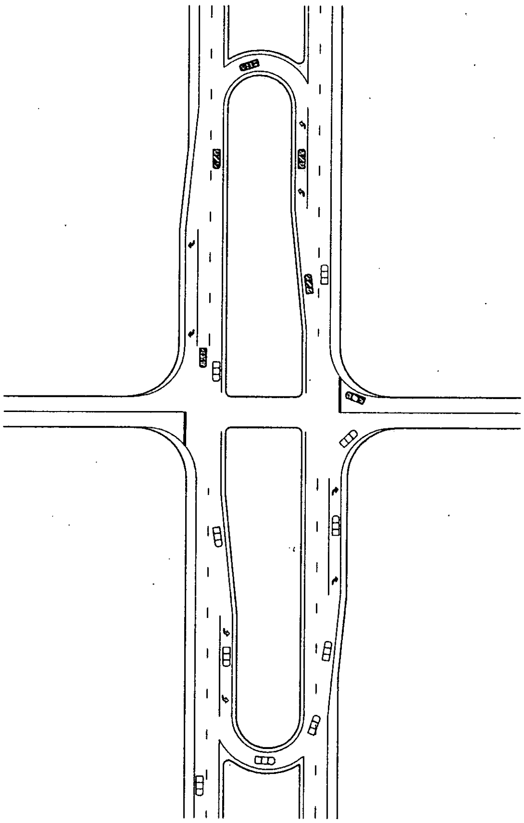

FIGURE 4 Indirect left-turn design.

A recent survey of state highway departments conducted by the Nebraska Department of Roads indicates that most states build rural expressways with at-grade intersections. When accident problems are found at these intersections, most states consider some type of signalization or signing improvement. One of the more novel corrective measures is the offset left-turn bay. This design attempts to minimize the sight distance blockage created by opposing left-turn vehicles. The blockage becomes more restricted with increasing median width. At present there has been no substantial research conducted on the safety benefits of offset left-turn paths; however, at least one-third of the state highway departments have successfully used this design at selected locations.

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REFERENCE

1. *Manual on Uniform Traffic Control Devices.* U.S. Government Printing Office, Washington, D.C., 1989.

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Grade-Separated Intersections

JOEL P. LEISCH

"Grade-separated intersections" refers to the various means of significantly increasing the capacity or resolving physical constraints by grade-separating the through movements on two intersecting roadways and interconnecting the two with ramps or roadways that form one or more intersections. Concepts and designs are discussed that have been successful in resolving intersection capacity problems and geometric or physical constraints in a manner responsive to the various needs and requirements of individual locations. Guidance is provided to the planner, designer, or traffic engineer in selecting the appropriate forms for a given condition. The 1990 AASHTO *Policy* discusses interchanges in Chapter X, but does not discuss adaptation of interchange forms to arterial (nonfreeway) situations. The categories of interchanges discussed include compact diamond, partial cloverleaf, and rotary forms. Each interchange type is described, and operational and design characteristics are discussed and compared with the others. The characteristics described, though generalized, reflect experiences gained through operational and design observation. All the interchanges presented are good forms when implemented under the site conditions that fit the specific design.

The increase in traffic on arterials in metropolitan areas has, in many cases, dictated the need to develop and implement solutions with higher capacity than can be provided by at-grade intersections. Also, there are some situations where capacity does not control but physical constraints may dictate the configuration of an intersection requiring a grade separation or other solution.

"Grade-separated intersections" in this paper refers to the various means of significantly increasing the capacity or resolving physical constraints by grade-separating the through movements on two intersecting roadways and interconnecting the two with ramps or roadways that form one or more intersections. The concepts and designs discussed here relate to experiences throughout the United States that have been successful in resolving intersection capacity problems and geometric or physical constraints in a manner responsive to the various needs and requirements of individual locations.

The 1990 AASHTO *Policy* (1) discusses interchanges at great length in Chapter X. However, there is no discussion related to adaptation of interchange forms to arterial (nonfreeway) situations. This paper is intended to supplement the *Policy* and provide guidance to the planner, designer, or traffic engineer in selecting the appropriate forms for a given condition.

WARRANTS/GUIDELINES

There are three controls that may dictate the need for a grade separation between two intersecting highway facilities: traffic volumes/capacity, safety, and alignment and profile (terrain).

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Traffic Volumes

In urban areas, traffic volume usually dictates the need for a grade separation where in the past an at-grade intersection even with improvements accommodated the through and turning traffic movements on the intersecting facilities. In most situations, volumes that create delay per vehicle approaching or in excess of 60 sec (LOS E-F) may be candidates for a grade separation to increase capacity and raise level of service (decrease delay). Obviously, other factors may influence implementation of a grade separation, such as improvement priorities and funding, not to mention physical constraints and environmental considerations.

Safety

In urban or rural areas, safety considerations may influence the need for a grade separation where an at-grade intersection exists. Where right-angle collisions (which may be related to traffic signal or stop control violations) are common, a grade separation could lower accident experience. Similarly, where a particular turning movement (usually a left turn) has particularly high accident experience, a grade separation may be used as a means of reducing conflicts and improving safety. The types of accidents associated with turning movements are usually sideswipe, rear-end, right angle, and head-on.

Terrain

The physical environment could influence the need or desirability of grade separations. They are already common in cities, such as in San Francisco, and in rural areas where the terrain and highway/street network require such a solution. Often the significant profile differences between the two intersecting facilities may require a grade separation as an economical solution. Interconnecting the two facilities to provide for traffic movements is often accomplished through other elements of the highway/street network.

Other physical or man-made constraints, such as railroads, rivers, and so forth, may also dictate the need to grade-separate two facilities that under "normal" circumstances could intersect and accommodate the traffic volumes through signal or stop control.

GRADE-SEPARATED INTERSECTION (INTERCHANGE) FORMS

The different forms of grade-separated intersections (interchanges) can be categorized in the following way: compac diamond, partial cloverleaf (Parclo), and rotary. The forms are depicted in Figures 1, 2, and 3, respectively.

Diamond Forms

Diamond forms are generally of three types: the single-point diamond, the compressed diamond, and the three-level diamond (see Figure 1).

The single-point diamond has the following characteristics:

- It takes little ROW.
- It has moderate capacity.
- It is a single intersection with three-phase signal control. Four-phase signal control is required if ramp through traffic movements are provided.
- It is the second most costly to construct of all diamond forms.
- Access is eliminated for a minimum of 1,500 to 2,000 ft along the priority facility.
- It is possible to have access on ramps if they are judiciously located.
- U-turn loops to interconnect ramps and reduce intersection traffic can be provided.
- Large left-turn radii can facilitate truck movements.

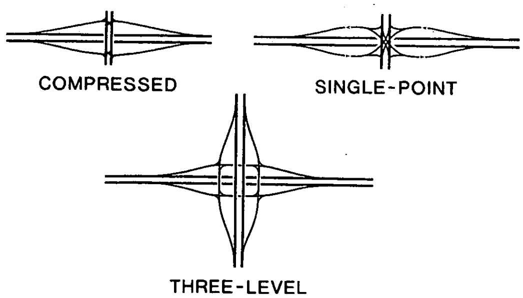

FIGURE 1 Diamond interchange forms.

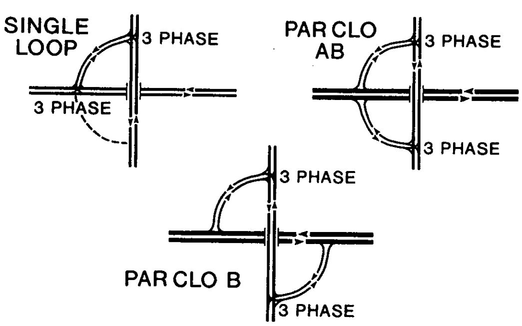

FIGURE 2 Partial cloverleaf forms.

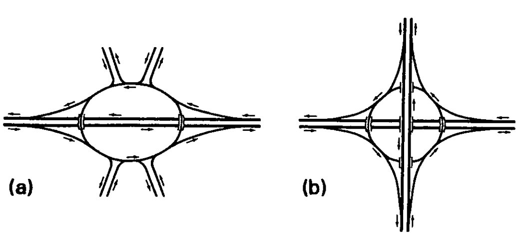

FIGURE 3 Rotary interchange forms.

The compressed diamond with ramp terminal intersections 200 to 400 ft apart has some characteristics that are similar to and others that are much different from the single-point diamond:

- It takes a moderate amount of ROW.
- It has moderate capacity.
- It has two intersections with coordinated four-phase overlap signalization.
- It is less costly to construct than the single-point diamond (generally 10 to 20 percent).
- Access is eliminated for a minimum of 1,500 to 2,000 ft along the priority facility.
- It is possible to have access on ramps if they are judiciously located.
- U-turn loops to interconnect ramps and reduce intersection traffic can be provided, or U-turns through intersection are possible.

The three-level diamond has very different characteristics from the other diamond forms:

- It takes the most ROW.
- It has high capacity.
- It has four intersections, each with two-phase signal control.
- It is the most costly to construct—nearly double the compressed diamond.
- Access is eliminated for a minimum of 1,500 to 2,000 ft along both facilities.
- It is possible to have access on ramps if they are judiciously located.
- There is no need for U-turn loops.

The application and implementation of one of these diamond forms is obviously related to the site-specific requirements associated with traffic/capacity, ROW, other physical constraints, and access needs.

Partial Cloverleaf Forms

Partial cloverleaf forms are also of three general types, as shown in Figure 2. They include a single-loop and two two-loop varieties. Their characteristics are somewhat different from the diamond forms. Partial cloverleaf forms often have application in locations where physical requirements, ROW constraints, access needs, and highway/street network configurations govern.

The single-loop or "cutoff" roadway form can have many applications in situations where turning traffic movements are not high and when roadway network and access requirements are compatible. This form is often applied where terrain controls and the two-way cutoff roadway is sufficient to accommodate the turning movements between the intersecting roadways.

- It takes little ROW.
- It has low to moderate capacity.
- It has two intersections—one on each roadway with three-phase signal control or stop control.
- It is generally not as costly as the diamond forms.
- Access is easy to coordinate; access off loop is possible.
- Consideration of highway/street network is required and could be used.
- It is the lowest in cost of the partial cloverleaf forms.

The Parclo B, two-loop/opposite quadrant form, is somewhat higher in capacity and responds to different traffic pattern and highway network requirements.

- It takes more ROW than the single-loop form.
- It has moderate to high capacity.
- It has free flow on the priority street, with right in and right out at connecting roads. Crossroad has two three-phase signal-controlled or stop-controlled intersections.
- It has construction costs similar to those of the two-loop/opposite quadrant form.
- Access is easy to coordinate; access from both loops is possible.
- Consideration of highway/street network is required and could be used.

Rotary Interchange Forms

Rotary interchanges have limited application. There are two types of rotary interchanges, as shown in Figure 3.

The first configuration (A) may be fitting in suburban areas where a major arterial serves a residential or partly commercial area with multiple streets forming five or more intersection legs and where traffic volumes are of the order that can be accommodated on a series of short weaving sections. The characteristics of a rotary interchange are as follows:

- The ROW required is about the same as or slightly more than the three-level diamond.
- It has moderate capacity.
- It has multiple intersections that could operate with yield control and with weaving between them.
- Its construction cost is similar to that of the compressed diamond.

An application of the rotary interchange is shown in Figure 3B in which two arterial highways have all through movements separated, using five structures. Each left-turning movement weaves with the other left-turning movements in negotiating three of the four weaving sections. A rotary with a radius of 400 to 500 ft produces weaving sections about 300 to 400 ft in length. The latter occupy an area approximately equal to a cloverleaf with 150-ft radius loops. In terms of serviceability, each weaving section is limited to a weaving volume of 1,200 to 1,500 vph. Construction cost approaches that of a three-level diamond.

SUMMARY

The interchange (grade-separated intersections) forms that have been presented in this paper have been implemented under varying traffic and physical conditions in urban, suburban, and rural areas where two or possibly more streets/highways intersect. The characteristics described, though generalized, reflect experiences gained through operational and design observation. All the interchanges presented here are good forms when implemented under the site conditions that most appropriately fit the specific design.

REFERENCE

1. *A Policy on Geometric Design of Highways and Streets*. AASHTO, Washington, D.C., 1990.

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Coordination of Basic Intersection Design Elements: An Overview

Ross J. Walker

The complexity of intersection design can vary from that of a simple rural location to a major intersection in a dense urban setting. However, even with the simplest location, many conflicting requirements must be balanced against each other to produce a safe and efficient design. Intersections are intended to operate where vehicles often must share space with other vehicles and pedestrians. Negotiating an intersection requires many simultaneous or closely spaced decisions, such as selection of the proper lane; maneuvering to get into the proper position; need to decelerate, stop, or accelerate; and need to select a safe gap. Five basic areas should be reviewed in conjunction with these decisions to produce a satisfactory design: intersection angle; coordination of the vertical profiles of the intersecting roads; coordination of horizontal and vertical alignment for intersections on curves; improvement of operation, safety, and capacity through channelization; and drainage requirements for safe operation. Not only must the horizontal layout be carefully thought out, but the coordination of the vertical and horizontal alignment should be given more emphasis. Poor integration of these two elements often results in an intersection that is less safe and uncomfortable to use. A number of features are discussed that could be used to improve the design. With the proper coordination, the intersection will give the user a safe, comfortable, easy-to-follow layout that allows for the limitations of the people using the facility while supplying an adequate level of service in an economical manner.

The complexity of intersection design can vary from that of a simple rural location to a major intersection in a dense urban setting. However, even with the simplest location, many conflicting requirements must be balanced against each other to produce a safe and efficient design.

The basic elements that must be taken into consideration fall into four categories: human factors, traffic considerations, physical elements, and economic factors. Human factors include driving habits, ability to make decisions, driver expectancy, decision and reaction time, conformance to natural paths of movement, and pedestrian use and habits. Traffic considerations include capacity, volumes, size and mix of vehicles, variety of movements, vehicle speeds (design speed and operating speed), and safety. Physical elements include character and line of abutting property, horizontal alignment, vertical alignment, available sight distance, intersection angle, conflict area, geometrics, traffic control devices, lighting, safety features, bicycle traffic, environmental impact, and drainage requirements. Economic factors include costs of improvements, effects on adjacent property (businesses) (i.e., raised median access, etc.), and impact on energy.

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Many of these factors have been studied over the years and require further study. This paper provides an overview of the basic intersection design elements.

The essence of good intersection design requires that the physical elements be designed to minimize the potential conflicts among cars, trucks, buses, bicycles, and pedestrians. In addition, the human factors of the drivers and pedestrians must be taken into account while keeping the costs and impacts to reasonable levels.

Intersections are intended to operate where vehicles often must share space with other vehicles and pedestrians. Negotiating an intersection requires many simultaneous or closely spaced decisions, such as selection of the proper lane; maneuvering to get into the proper position; need to decelerate, stop, or accelerate; and need to select a safe gap.

The horizontal aspects of intersection design have received a great deal of attention, whereas the vertical elements have been reviewed to a much lesser extent. This paper explores the coordination of these two basic elements in such a way that the human factors, traffic considerations, and economics are integrated into the design. This will produce an intersection that is safe, comfortable, and convenient for all users while providing an adequate level of service.

Five basic areas should be reviewed in conjunction with the above to produce a satisfactory design: intersection angle; coordination of the vertical profiles of the intersecting roads; coordination of horizontal and vertical alignment for intersections on curves; improvement of operation, safety, and capacity through channelization; and drainage requirements for safe operation.

The author was involved in developing many of the figures used in the Transportation Association of Canada's *Manual of Geometric Design Standards* (*1*). A number of these have been modified and used below to illustrate the various overview features in this paper.

INTERSECTION ANGLE

Figure 1A shows a simple 90-degree angle and a skewed angle intersection. The 90-degree angle provides the best operation. However, this is not always possible to achieve, and the designer has to then deal with the skewed angle.

As the angle varies from 90 degrees, a number of problems arise:

1. The area of conflict increases, as shown in Figure 1B.
2. Visibility is limited—drivers entering the intersection have difficulty seeing approaching traffic. When trucks turn

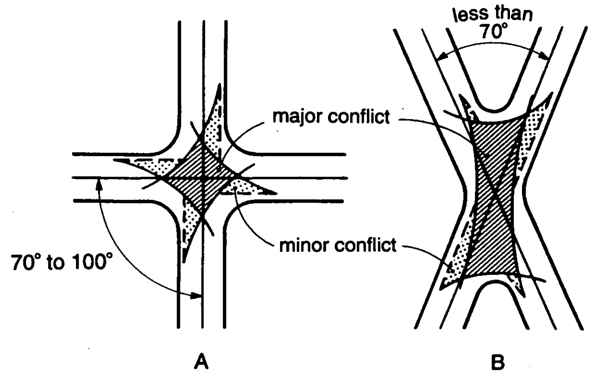

FIGURE 1 (A) Right- and (B) acute/obtuse-angle intersections.

through an obtuse angle, the driver may have a blind area to the right of the vehicle.

3. Larger turning roadway areas are required for trucks (*2,3*).

4. The exposure time through the intersection is increased. This is most critical for trucks due to their slower acceleration and for pedestrians who are unable to walk quickly.

All of this increases the potential for accidents.

AASHTO (*4*) indicates intersection angles of between 60 and 120 degrees, whereas the Canadian Transportation Association's manual (*1*) limits this to 70 to 110 degrees. Every attempt should be made to keep the angle as close as possible to 90 degrees. However, where costly or severe constraints occur, angles as low as 60 degrees are acceptable. New construction should not include skewed angles less than 60 degrees without special design and control features to mitigate the effects of the skew. These may include positive traffic control such as stop, or traffic signals. Adequate corner sight distance and extra pavement area for trucks to maneuver so that they can see oncoming traffic would have to be ensured.

IMPROVEMENT OF OPERATION THROUGH REALIGNMENT OF THE MINOR ROAD

Figures 2A to 2D show realignments to improve the skew angle and the operation through the intersection. The selection of the appropriate type of treatment will depend on adjacent property restrictions. Sufficient decision and stopping sight distance, in accordance with AASHTO (*4*), should be maintained, with special attention if the intersection is on a vertical curve. Care should also be taken to avoid too short a radius on road curves approaching the main road. The radii of these curves will depend on the design speed of the approach roadway. The first curve should be flatter than the second to provide the driver with a safe speed transition. Advance warning signs may be needed to alert the motorist, who may not anticipate the change in direction.

Where realignment as shown in Figures 2A and 2B is not possible, it may be necessary to use offset intersections.

Figure 2C shows an undesirable offset solution. Traffic must turn left off the main road a short distance after entering it. If it is necessary to use this arrangement, a left-turn bay, with

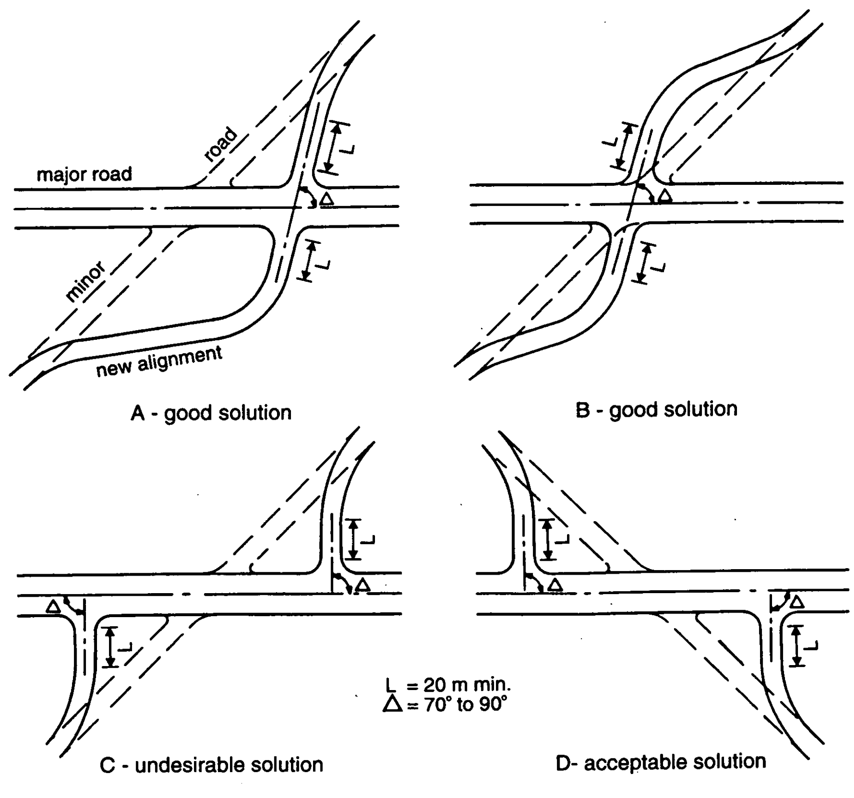

FIGURE 2 Realigned intersections.

provisions for the main road to safely bypass the left turning vehicle, should be included in the design.

Figure 2D shows an acceptable solution because it allows for a left turn onto the main road and then a right off the main road. It would also be desirable to provide a right-turn lane to reduce the conflict between the slow-moving right-turn vehicles and the high-speed through traffic on the main road.

Figures 2C and 2D are mostly applicable to rural situations. There will also be cases in which existing jogs in the urban street system will have to be eliminated to reduce congestion. In this case traffic signals would provide the control needed for the flat intersection angle.

COORDINATION OF VERTICAL PROFILES THROUGH INTERSECTIONS (BOTH ROADS ON TANGENT)

In many instances, the crown of the main road is carried through the intersection, forcing the minor road traffic to drive over the crown as shown in Figure 3A. Where the two roads are on relatively flat grades, this may be acceptable. However, in areas where traffic does not have to stop or when traffic signals are used, the minor traffic has a tendency to go over the hump at higher speeds, which increases the accident potential.

This problem is accentuated when the cross road is on a grade, as shown in Figure 3A. Here the minor road profile has been adjusted to fit the crown of the mainline. This is not a particularly good solution because it requires careful design of the sag curves to ensure that the passage over the crown is not hazardous if driven too quickly. If proper K values, adequate vertical curve lengths, and tangents are used, it will operate reasonably well. This design usually results in a costly solution. A more desirable alternative is the design shown in Figure 3B. It provides a much smoother profile. Here the major road has a reverse crown, so that it slopes in the same direction as the minor road. This will not affect the operation on the major road, but it will take the roller coaster effect out of the minor road profile. This is also a more economical solution and, with careful design, it will fit most situations.

COORDINATION OF HORIZONTAL AND VERTICAL ALIGNMENT FOR INTERSECTIONS ON CURVES

Intersections on curves should be avoided because sight distance is often restricted and turning traffic has to deal with the superelevation. Where possible, the minor road should be realigned to intersect beyond the curve. When it is necessary to have an intersection on a curve, there are a number of ways to address the problem. Figure 4A shows the easiest situation, where the grade of one road is in the same direction as the superelevation of the cross street. In this case, joining the two profiles is relatively easy.

What happens when the grade and superelevation are reversed? This often results in a roller coaster grade (Figure 4B), which can again cause operational and safety problems. K values and tangent distances on each side of the intersection as shown in Figure 4B have to be chosen to provide a smooth

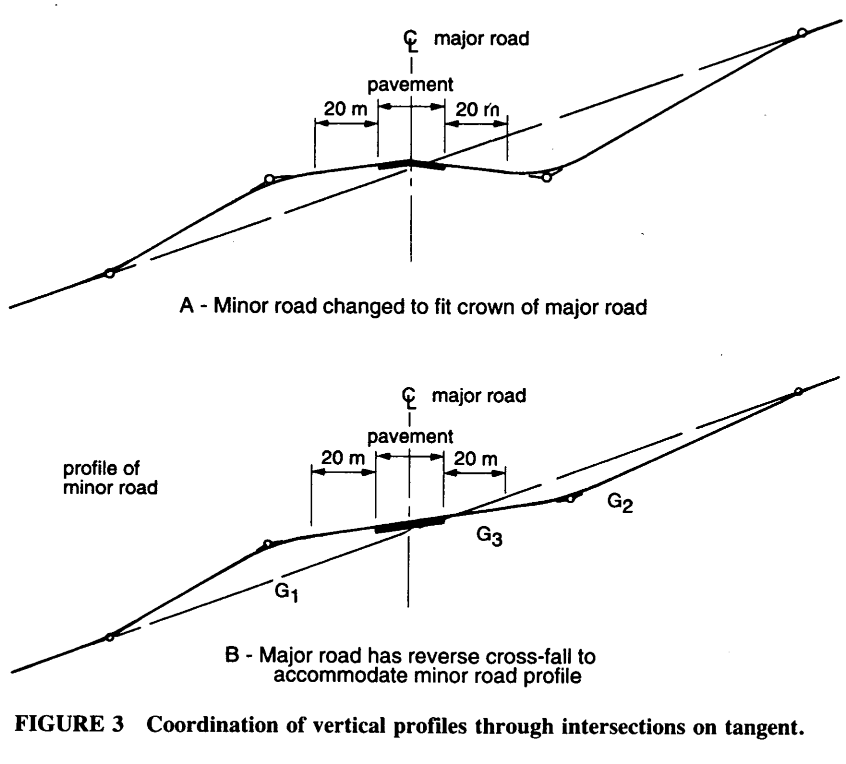

FIGURE 3 Coordination of vertical profiles through intersections on tangent.

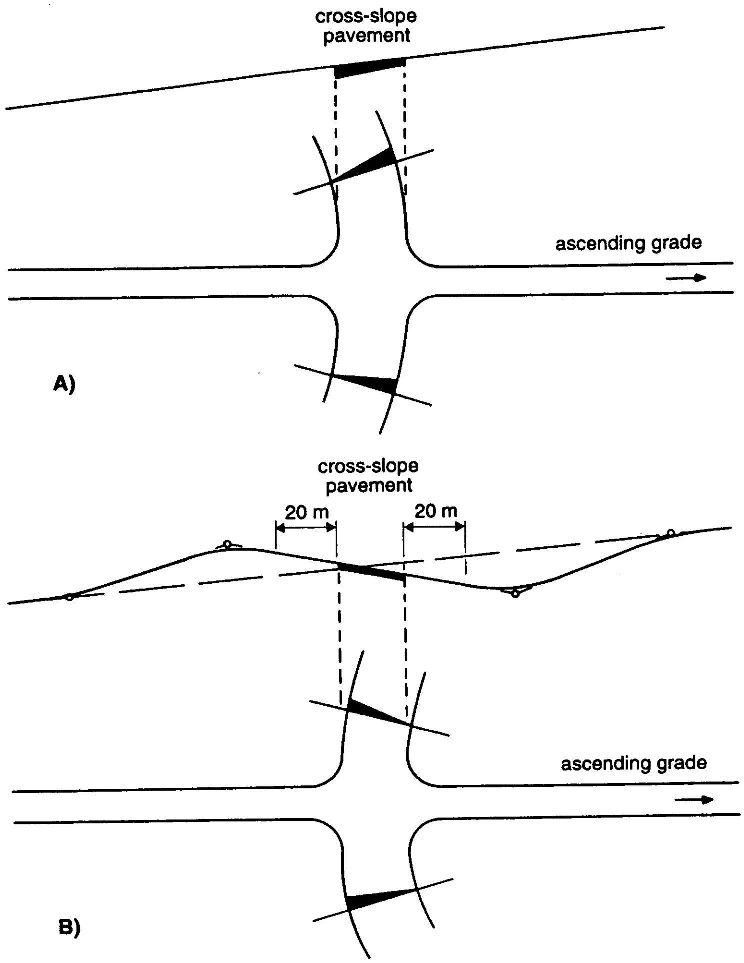

FIGURE 4 Coordination of horizontal and vertical alignments on curves.

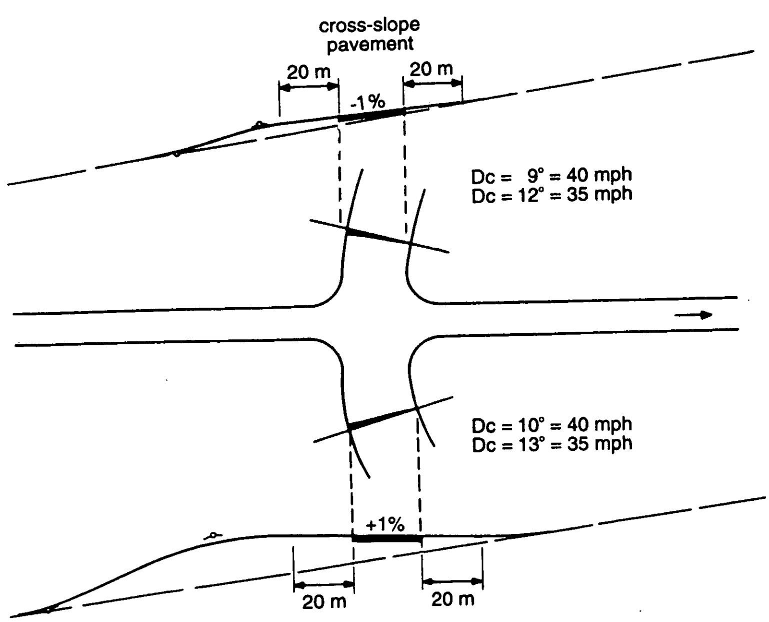

FIGURE 5 Combining horizontal and vertical design.

design. This may not always be possible due to physical limitations of the site.

This is an area where more care is needed to coordinate the horizontal and vertical alignment. The vertical alignment can be improved on the cross street if less superelevation is used on the curve. Two possible solutions are shown in Figure 5. In the lower diagram the superelevation has been reduced to +1 percent and in the upper to −1 percent. This can only be done if the degree of curve, the design speed, and resulting side friction factor allow it.

In urban settings, speeds are lower, and drivers are interrupted by traffic signals and will accept a higher level of side friction than in rural areas. AASHTO has provided friction factors for intersection design for speeds from 30 to 40 mph. Using these values in conjunction with varying superelevation rates, a series of curves for various design speeds can be used as shown in Figure 6.

For the example shown in heavy dashed lines in Figure 6, the 200-m-radius (approximately 9-degree) curve results in a 70-km/hr (43-mph) design speed with a superelevation of 2.3 percent.

By comparing Figures 4 and 5, the improvements to the grade on the cross street can readily be seen. Figure 6 is very useful for intersection design in urban areas, where drivers are willing to accept a higher side friction factor. In this case, Method 2 of AASHTO (*4*) is being used where side friction is used before superelevation. These curves are not recommended for rural situations, where speeds are much higher and drivers will not accept the higher friction factors.

IMPROVEMENT OF OPERATIONS SAFETY AND CAPACITY THROUGH CHANNELIZATION

The *Intersection Channelization Design Guide* (*5*) provides a complete analysis and description of good channelization design practice. Figures 7 to 9 show, in incremental steps, some of the features and advantages of channelization. Each layout, starting with a simple turn bay of Figure 7A, provides increased capacity, ease of operation, and safety. Channelization must be easy and natural to follow. Too many islands, or islands improperly placed, cause confusion. Each of these designs is simple and easy to follow. Selecting a particular design for any location will depend on the space available and conditions present at the intersection. Considerations for pedestrians, bicycles, bus stops, and truck types will also affect the final layout.

Intersections on rural roads can often be very hazardous, especially if there are a significant number of left turns. If no provision has been made for a left-turn bay, the through traffic has to stop. Even if there is adequate sight distance, rear-end collisions will occur. The simple flush (i.e., no curbs) left-turn layout shown in Figure 7A improves the safety aspects and operation because it provides a bypass lane for the high-speed through traffic and thus reduces the number and severity of rear-end accidents. The through traffic does not have to slow down or stop for the left-turn vehicle.

The left-turn bay shown in Figure 7A is offset from the centerline and therefore is not in line with the left-turn traffic approaching from the opposite direction. The bay for the

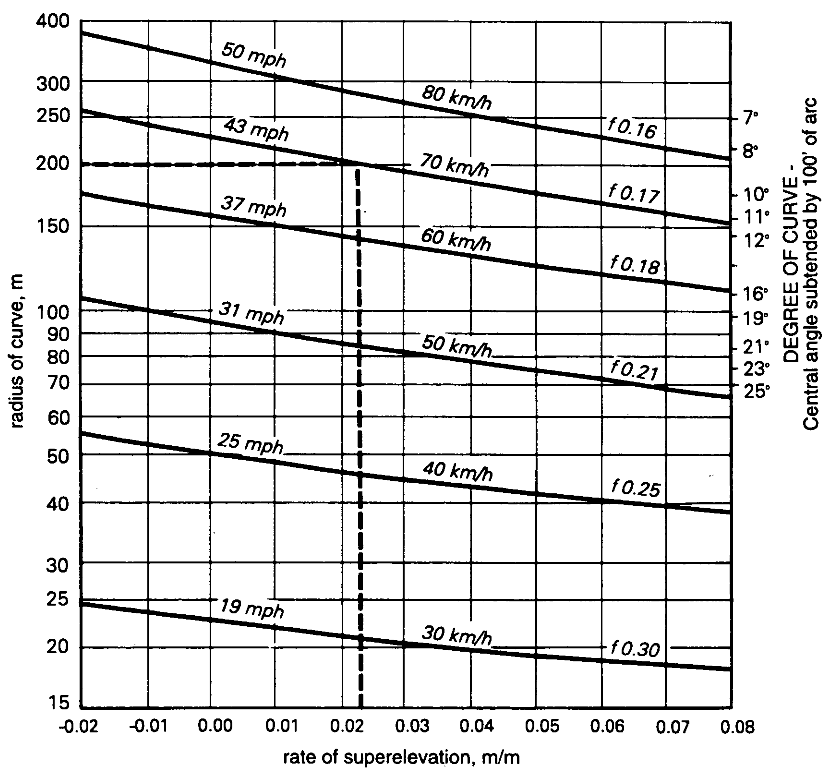

FIGURE 6 Relationship of speed, radius, and superelevations at urban intersections.

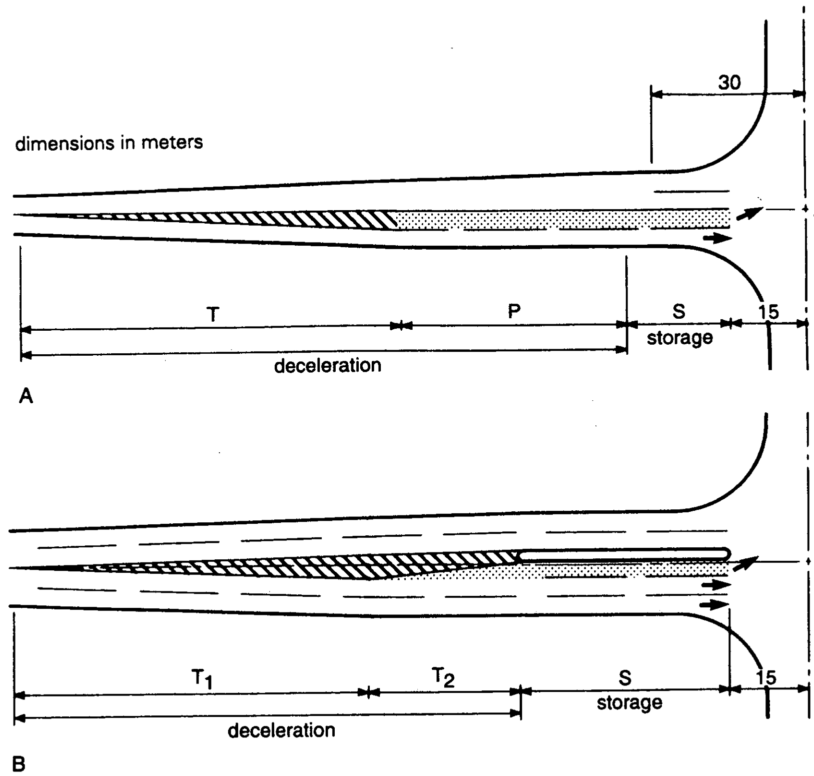

FIGURE 7 **(A) Introduced flush left-turn lane and (B) left-turn lane, introduced raised median.**

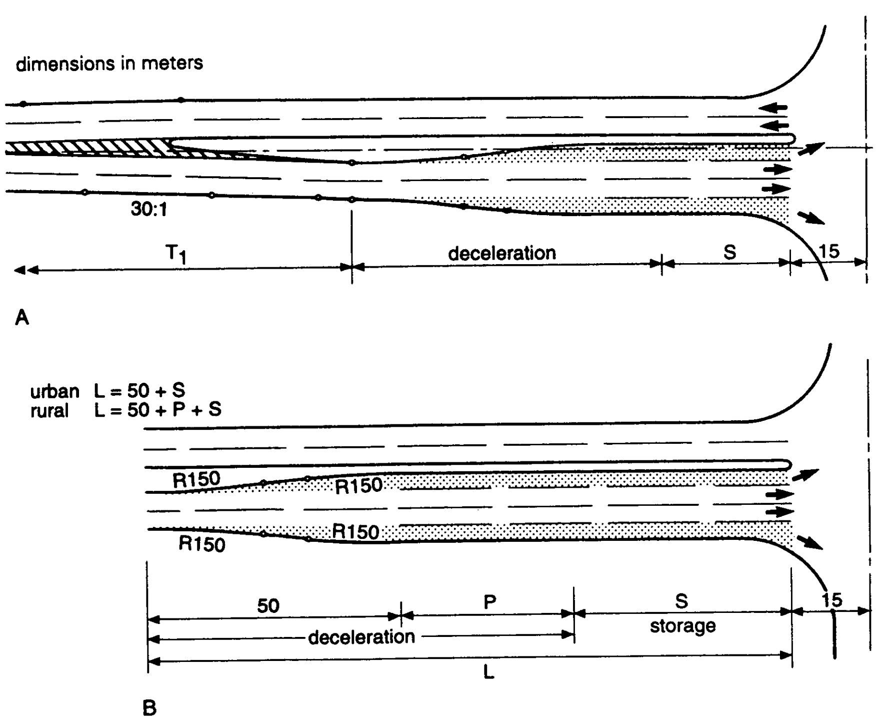

FIGURE 8 **(A) Turning lane design, introduced median, and (B) continuous raised median.**

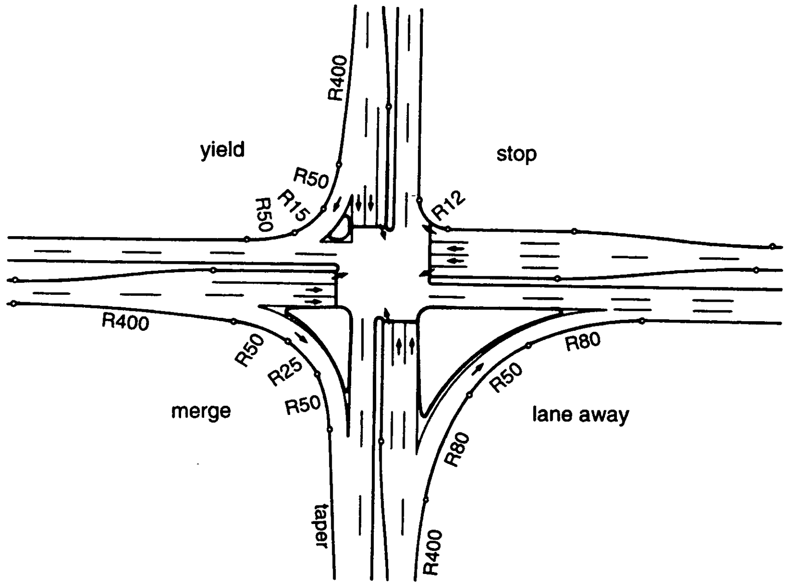

FIGURE 9 Typical right-turn lane channelization.

other direction is also offset from the centerline, which provides for safer operation. In some urban situations where space is restricted, the left-turn bays may not be offset but simply lined up with each other. This is an acceptable practice where speeds are lower and drivers are used to more constricted operations. However, wherever possible, the offset should be maintained because it provides for freer and safer flow of traffic. The layout allows for a deceleration distance made up of a taper (t) to shift the through traffic to the right and a parallel p distance or extension of the bay to safely complete the deceleration. After this, a space is left for storage.

Although Figure 7A is used in many rural situations, it does not provide any protection for the left-turning traffic. Figure 7B shows a more pronounced taper where the left turn is somewhat protected by the taper. In this case, all traffic is first shifted to the right and left-turn traffic has to shift back to the left, while the through continues on. Tapers for deceleration and storage lengths are provided in the design. The short raised median provides more guidance and protection for the left-turn vehicles. Some jurisdictions prefer this design because the raised portion of the median is offset from the main traffic flow and reduces the possibility of hitting the raised median. However, it does not completely protect the left-turning vehicle.

Figure 8A shows a full introduced median with both left- and right-turn lanes. Smooth tapered transitions have been provided for efficient traffic flow. This design gives more positive direction to vehicles, resulting in protection for left and right turns and smoother flow.

Figure 8B is basically the same as Figure 8A except that the median is continuous.

Four typical right-turn island layouts are shown in Figure 9.

- The top right corner has a simple radius and a stop condition.
- Top left shows a yield condition with a minimum three centered curve, which minimizes the property required while providing a turning path convenient for trucks. The 15-m (50-ft) radius assures that the entry angle will provide good sight distance, and if there is no traffic coming, vehicles will be able to commence accelerating on the curve with 50-m (150-ft) radius.
- The bottom left shows a tapered exit followed by an entrance merge.
- The bottom right has a tapered exit and a lane away (exclusive entering lane) at the entrance.

Both of these layouts provide for progressively higher right-turn volumes.

DRAINAGE

One of the most important features of intersection design is to ensure that proper drainage has been provided. This is especially true of channelized intersections on grades and curves. The designer must balance the horizontal and vertical alignment while making sure that no excess water gathers on the pavement surface, which could cause hydroplaning.

This requires careful design of pavement edge profiles by integrating the left- and right-turn islands with the horizontal and vertical curvature and grades. In doing this, the following points should be checked.

- Minimum grades on curb and gutter sections should be 0.5 percent.
- Additional catch basins should be placed in low areas to eliminate ponding.
- Cross-fall on each pavement surface should be checked through the transition areas from one superelevation to another to make sure that flat sections are kept to a minimum.

Minimum cross-fall of 2 percent is desirable to keep water from accumulating.

- The grades and cross-falls of both intersecting roads along with their right-turn lanes should be smoothly transitioned into each, allowing for all the preceding factors. This will often require that pavement edges be splined to effect a smooth profile. (Calculated grades will often give an uneven profile of the pavement edge. These can be smoothed with the use of a long plastic spline that is held down by weights. The spline passes above and below, close to calculated points, thus providing an even curved line. The elevations are then scaled off the large-scale profile.)
- When the cross section of the intersection is checked in both directions, it must provide a smooth path. There should be no surprises such as uneven operation caused by bumps or erratic changes in cross-fall.

Figures 10 to 14 are pictures of some existing intersections that illustrate the basic elements discussed above.

Figure 10 shows an intersection that has been placed just before the beginning of a horizontal curve. The superelevation

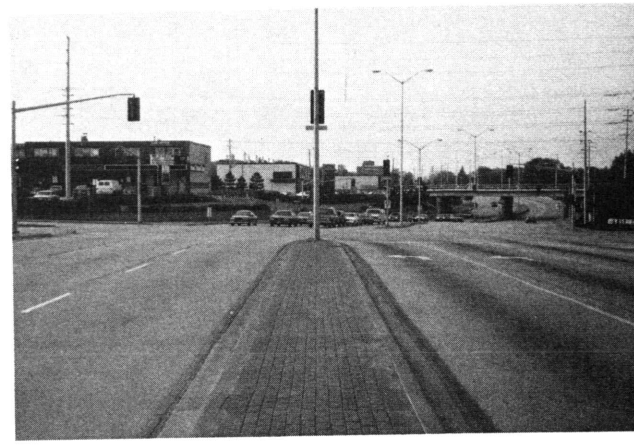

FIGURE 12 Major arterials intersecting on grades, looking northbound.

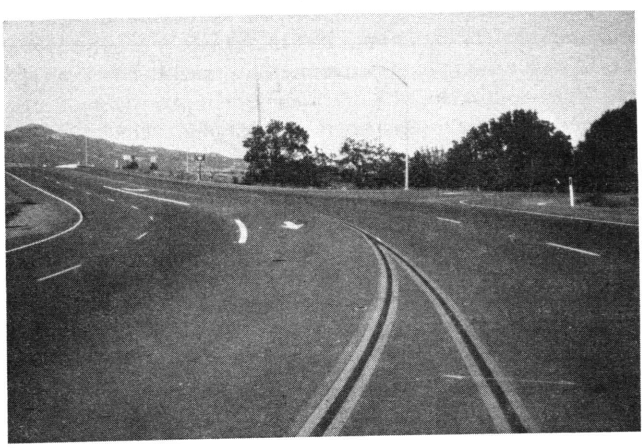

FIGURE 10 Intersection at beginning of horizontal curve.

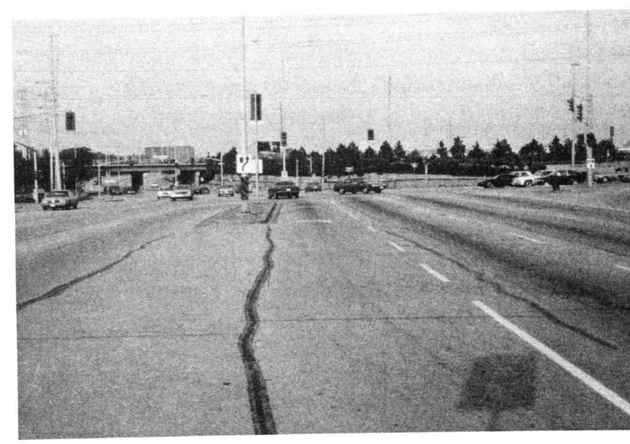

FIGURE 13 Major arterials intersecting on grades, looking eastbound.

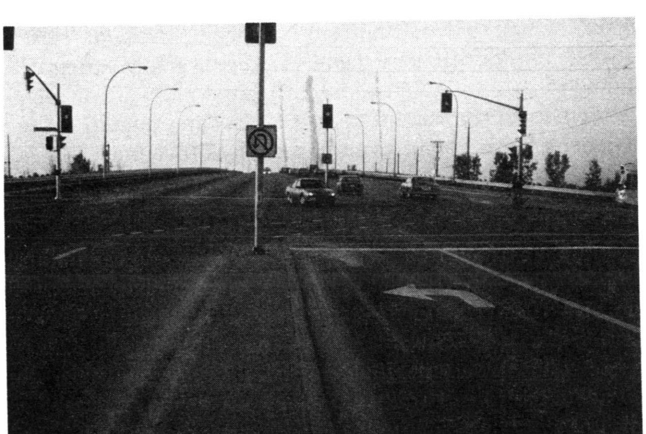

FIGURE 11 Mainline grade carried through the intersection.

FIGURE 14 Royal Avenue and 6th Street—roller coaster.

runoff has been taken through the intersection, and the cross street easily matches it. The flush left-turn lane bay is essential, because the intersection is placed just beyond a crest curve. Any vehicles waiting to make a left turn could create a hazardous condition for through traffic if the bay were not there.

Figure 11 provides an example of where the grade of the main road has been carried through the intersection. The grade is approximately 4 percent. The design provides a continuous grade through the intersection for both the major arterial and the cross street. The cross street is tilted to meet the grade of the main road. This location is in the snow belt, and the only movement that may have some trouble is the left turn against the grade in icy conditions. Because there are no abrupt changes in the grades lines, the whole intersection is smoothly developed and operates very well. Drainage patterns are also very good.

Figures 12 and 13 provide an example of two arterial streets intersecting on different grades and partially on horizontal curves. This intersection design was part of the need to grade-separate a main rail line, which crosses both streets very close to the intersections. The final layout had to allow for the closeness of the two grade separations while integrating all of the design elements (coordination of horizontal and vertical alignment, consideration of superelevation cross-fall through the intersection to allow for all directions of travel, minimum curb and gutter grades, and extra catch basins at low points to eliminate ponding and possible hydroplaning). Besides meeting these requirements, the fully channelized high-capacity intersection provides safe, convenient, and efficient operation.

Figure 14 has a roller coaster design, which could have been avoided with a little more care in the vertical design. The cross street's crown has been partially maintained, which has caused a dip on the through street as shown in the photograph. It would have required some special treatment on the lower side of the cross arterial to maintain the cross-fall, but it could have been achieved. Presently any driver approaching this intersection at more than 20 mph experiences discomfort. This type of operation is unsatisfactory for alert drivers, but for others it is a hazard. This intersection does not allow for human factors.

SUMMARY

The main factors that must be coordinated in intersection design were highlighted. Not only must the horizontal layout be carefully thought out, but the coordination of the vertical and horizontal alignment should be given more emphasis. Poor integration of these two elements often results in an intersection that is uncomfortable and less safe to use. A number of features have been discussed that could be used to improve the design. With the proper coordination, the intersection will give the user a safe, comfortable, easy-to-follow layout that allows for the limitations of the people using the facility while supplying an adequate level of service in an economical manner.

REFERENCES

1. *Manual of Geometric Design Standards for Canadian Roads*. Metric edition. Transportation Association of Canada, Ottawa, Ontario, Canada, 1986.
2. J. E. Hummer, C. V. Zegeer, and F. R. Harrison. Effects of Turns by Larger Trucks at Urban Intersections. In *Transportation Research Record 1195*, TRB, National Research Council, Washington, D.C., 1988, pp. 64–74.
3. D. B. Fambro, J. M. Mason, Jr., and N. S. Cline. Intersection Channelization Guidelines for Longer and Wider Trucks. In *Transportation Research Record 1195*, TRB, National Research Council, Washington, D.C., 1988, pp. 48–63.
4. *A Policy on Geometric Design of Highways and Streets*. AASHTO, Washington, D.C., 1990.
5. T. R. Neuman. *NCHRP Report 279: Intersection Channelization Design Guide*. TRB, National Research Council, Washington, D.C., 1985.

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Freeway and Interchange Design: A Historical Perspective

JOEL P. LEISCH

Controlled-access facilities for vehicular traffic first came into being during the 1920s. Since that time, their design has continually evolved as transportation professionals have gained increased experience in their operation, direction from research, and expanded knowledge of human factors related to driver characteristics and expectations. The need for such facilities was much as it is today—capacity and safety for highways to move people efficiently. Early controlled-access facilities, however, were predominantly isolated grade separations with roadways connecting the two grade-separated highways or streets. It was not until the 1930s that freeway or expressway facilities were constructed with grade separations and interchanges. By the late 1930s and early 1940s, freeways of significant length were constructed as part of a planned system of controlled-access facilities. A variety of different interchange forms came into being as well; the cloverleaf, diamond, and trumpet were the predominant types. By the late 1950s, every basic interchange form had been constructed. Although those basic types have not changed, geometric variations have been developed, constructed, and operationally tested. Discussed are the development and evolution of freeway and interchange design and the safety, operational, and human factors research over the last 30 years that has contributed to recognition of the interchange forms and design elements that produce safe and efficient operations consistent with driver characteristics and expectations.

Controlled-access facilities for vehicular traffic first came into being during the 1920s. Since that time, their design has continually evolved as transportation professionals have gained increased experience in their operation, direction from research, and expanded knowledge of human factors related to driver characteristics and expectations. The need for such facilities was much as it is today—capacity and safety for highways to move people efficiently.

EARLY FREEWAY AND INTERCHANGE DEVELOPMENT

Early controlled-access facilities were predominantly isolated grade separations with roadways connecting the two grade-separated highways or streets. The first interchange was constructed in Woodbridge, New Jersey, in 1928 as a cloverleaf (see Figure 1). This was unique at the time because all through and turning movements were uninterrupted.

It was not until the 1930s that freeway or expressway facilities were constructed with grade separations and interchanges. The earliest of these were relatively short in length and were generally not conceived as part of a system or network of controlled-access facilities. Interchanges were predominantly of the cloverleaf type. Horizontal and vertical geometry and cross-sectional features were generally not distinguishable from other lower-type facilities. Some of these early facilities include the Merritt Parkway in Connecticut (Figure 2), the Henry Hudson Parkway in New York City, and Lake Shore Drive in Chicago (Figure 3).

By the late 1930s and early 1940s, freeways of significant length were constructed as part of a planned system of controlled-access facilities. A variety of different interchange forms came into being as well; the cloverleaf, diamond, and trumpet were the predominant types. During this period, several other countries built freeways. In Ontario, Canada, the Queen Elizabeth Way was constructed, and in Germany, the first of the Autobahns was built. In the United States, a long section of the Pennsylvania Turnpike was opened to traffic (Figure 4) and a section of the Davison Expressway with frontage roads was constructed in Detroit (Figure 5).

The Pentagon Road Network (Figure 6) was completed and opened as the first freeway network. This was a system of 10 mi of freeway with 21 grade separations and 11 interchanges. Commonly referred to as the Mixing Bowl, it later became a laboratory of operational experience and research that influenced changes in geometric design criteria for freeway facilities of the future.

The first freeway constructed in the western part of the United States was appropriately in Los Angeles-Arroyo Seco Parkway (Figure 7). It not only connected Los Angeles with Pasadena, but also channelized the Arroyo Seco (Dry Creek) as part of a flood control project. Several diamond interchanges were constructed (Figure 8) as an effort to conserve right-of-way.

POST-WORLD WAR II FREEWAY DEVELOPMENT

After World War II, considerable effort was directed towar planning, design, and construction of freeways and freewa networks in metropolitan areas. In 1944, the Interregiona Highway Commission, which was appointed by Presiden Roosevelt, released recommendations for construction of ap proximately 34,000 mi of freeways to connect all cities wit populations of 300,000 or more and 80 percent of cities c 50,000 or more. This later became the basis for the Interstat system of 1956. The 1944 Highway Act incorporated the com mission's recommendations with expansion to 40,000 mi bu failed to authorize special funding. Portions of this syste were built, however, using primary highway funding, with th

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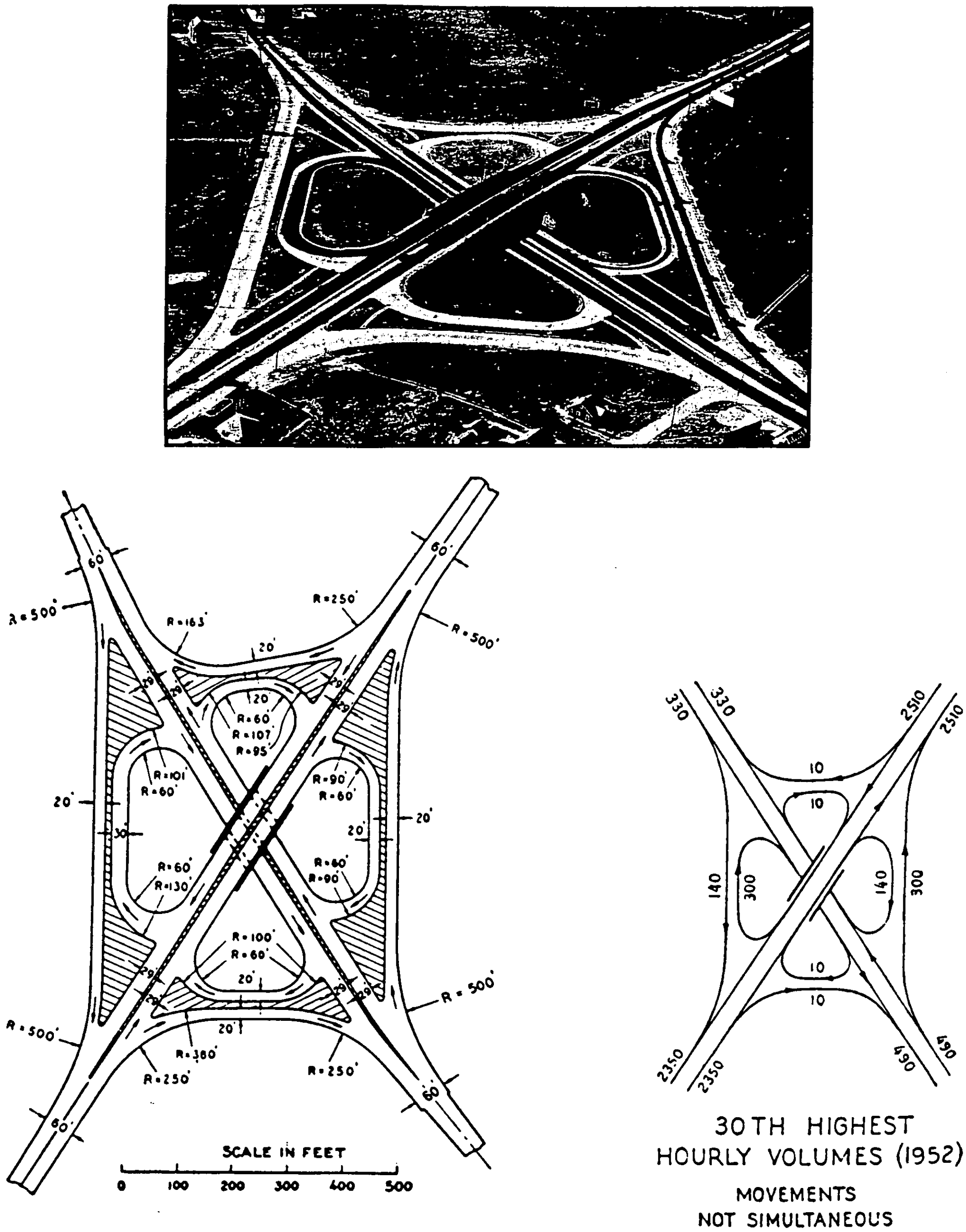

FIGURE 1 Cloverleaf interchange in Woodbridge, New Jersey (1928) (*1*).

ederal government and the states sharing the cost. Many of he facilities constructed during this period were conversions f 2-lane primary highways to 4-lane divided facilities. Many f these were the first to be reconstructed in the late 1970s nd 1980s because they were physically, operationally, and eometrically deficient. They had been designed using criteria f the 1940s, which was based on limited experience.

In 1956 the Federal Highway Act authorized funding at a 0/10 federal/state proportion, and the 42,500-mi system of nterstate highways was born. Finally, the financial resources ere in place to implement the greatest single public works roject of all time.

Although the funding resources were in place, the engi-eering resources to plan, design, and implement the system ere not. Few engineers had experience in freeway and interchange design in 1956. To accomplish the implementation of such a massive system in 20 years required extensive mobilization and training. As with mobilization for the second World War, public agencies and private enterprise responded in exemplary fashion. By 1965, some 27,000 mi of the system was constructed or was under construction—a monumental effort. By the early 1970s, more than 90 percent of the system was completed and opened to traffic.

By the mid-1960s, enough experience had been gained in operating the Interstate highway system and research conducted relating geometrics, operations, and safety that the design criteria of the 1940s and 1950s was being rethought. This naturally is part of the planning, design, and implementation process. The freeways of the 1940s, 1950s, and early 1960s also became a laboratory of observation and a research

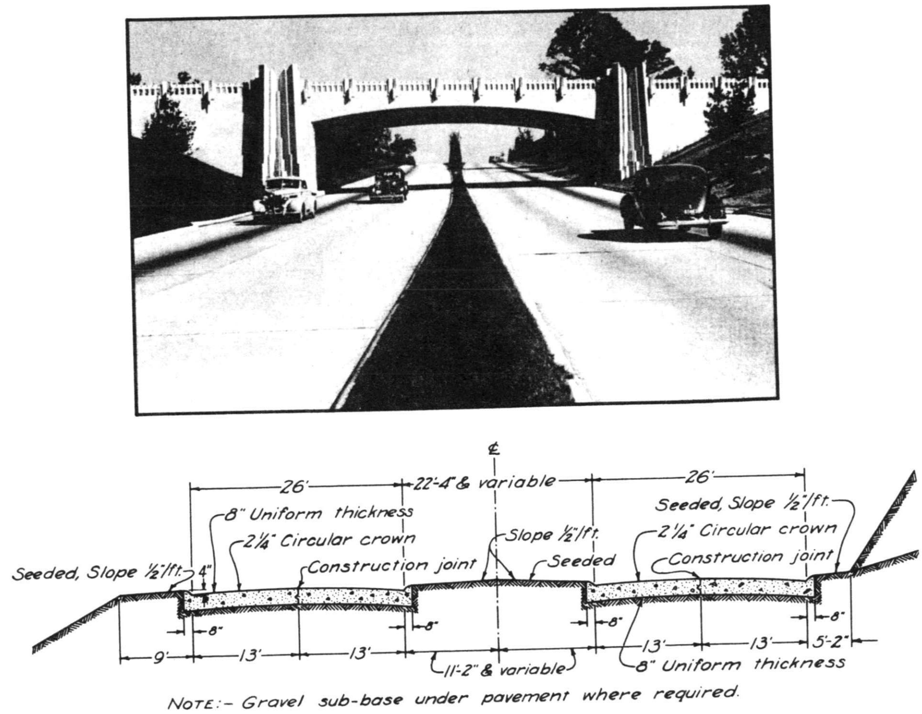

FIGURE 2 Photograph and typical cross section of Merritt Parkway, Connecticut.

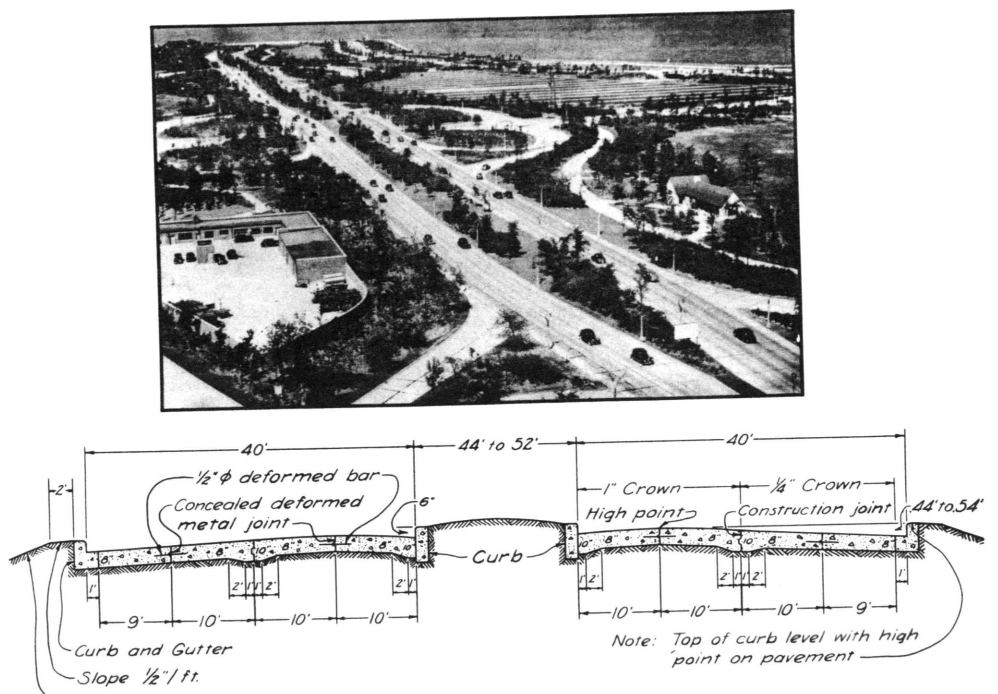

FIGURE 3 Photograph and typical cross section of Lake Shore Drive, Chicago.

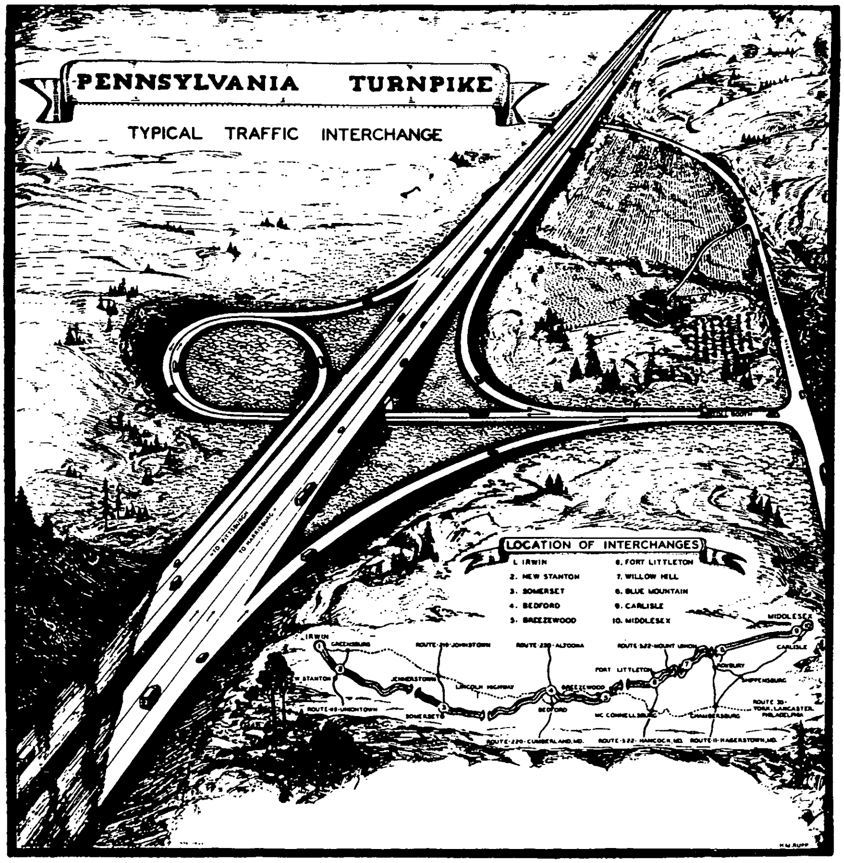

FIGURE 4 Pennsylvania Turnpike.

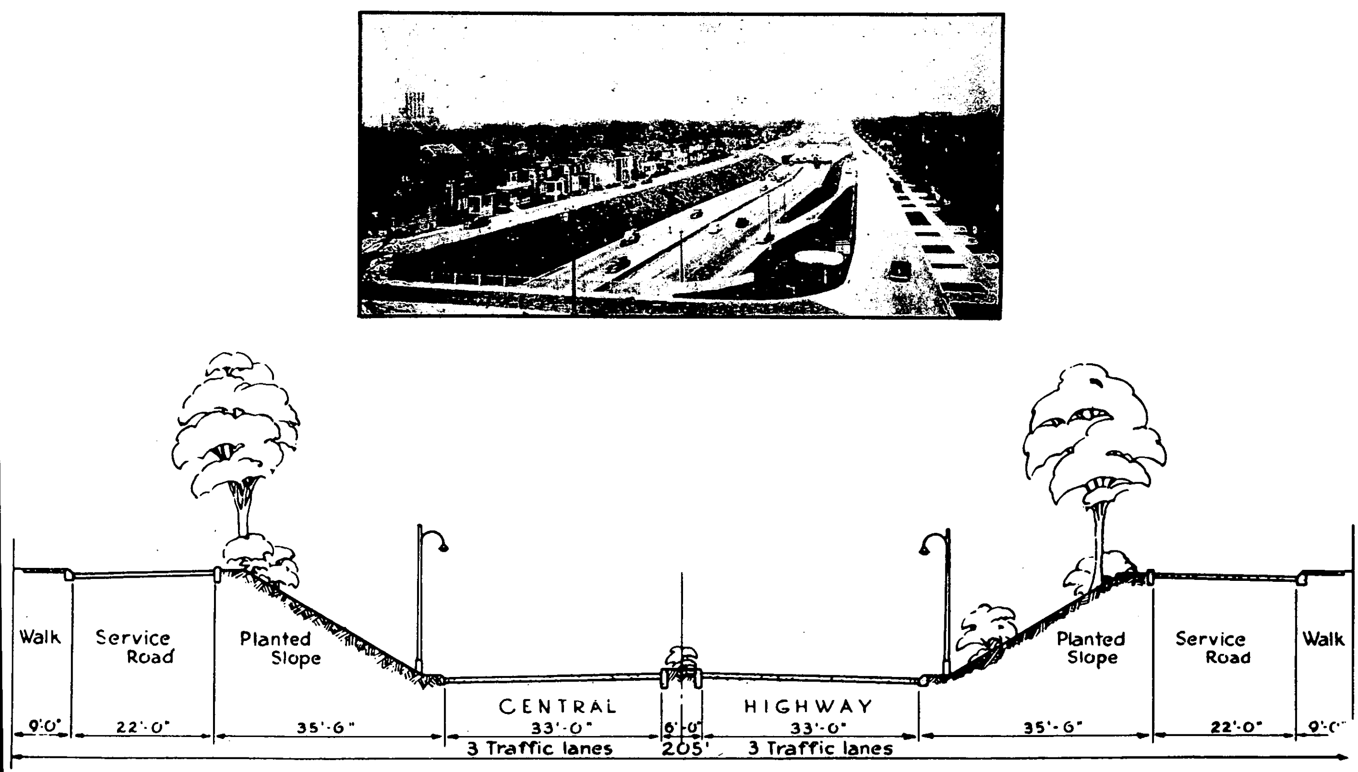

FIGURE 5 Photograph and typical cross section of Davison Limited Freeway, Detroit, Michigan.

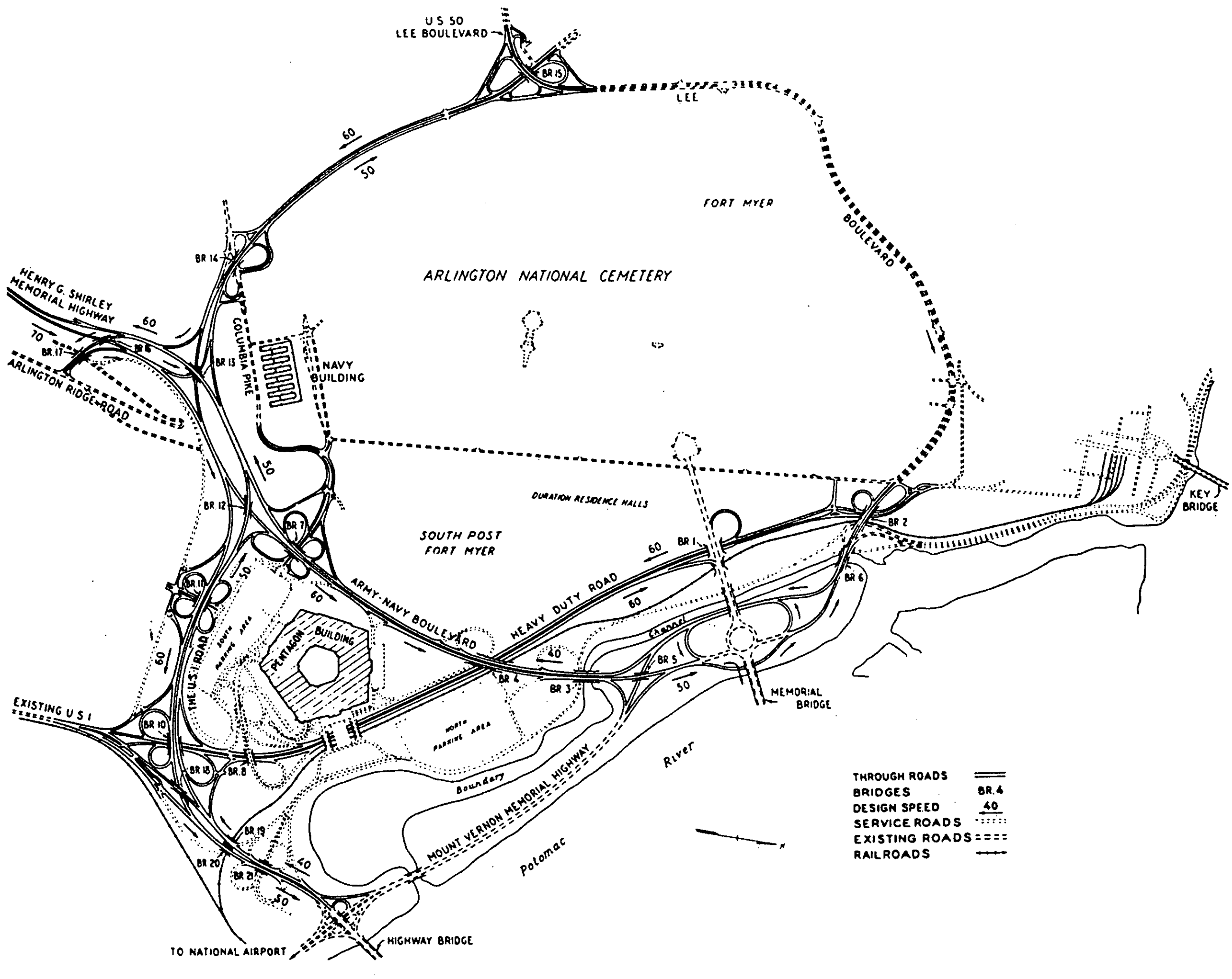

FIGURE 6 Pentagon road network.

resource. The consequences of the operational experience and the research was the foundation for changes in the future AASHO and AASHTO design policies, the *Highway Capacity Manual*, and the *Manual on Uniform Traffic Control Devices*. Countless other publications have been developed or modified since the mid-1960s; they reflect the experience gained and the research accomplished.

The remainder of this paper will discuss freeways and interchanges and the evolution of their design and operation and give some direction for the future.

EVOLUTION OF GEOMETRIC DESIGN OF FREEWAYS AND INTERCHANGES

In general terms, a freeway has always been a high-speed highway accommodating large volumes of traffic. According to the AASHTO definition, a freeway is an expressway with full control of access (i.e., a divided arterial highway with all intersecting roads grade separated and all entrances and exits accomplished by high-speed merging and diverging maneuvers). To achieve both high vehicular volumes and high speed we have learned that the standards of the freeway must be of superior caliber. The freeway is also characterized by efficiency and safety, a by-product of control of access and high design criteria.

The aspects of freeway design covered here are those that relate primarily to geometric features. The discussion, therefore, is oriented toward geometric design and planning considerations. Geometric design is the dimensional design of a highway; it may also be defined as the design of the visible dimensions of a highway with the objective of forming (shaping) the facility to the characteristics and behavior of drivers, vehicles, and traffic. Geometric design is a dynamic area of highway and traffic engineering which, in its true sense and broad application, translates research and operational experience into a physical highway plant.

Geometric design of freeways involves features of location, alinement, profile, cross section, and interchanges. The form and dimensions of the freeway, made up of these elements, should properly reflect driver safety, desire, comfort, and convenience. Closely related and considered in geometric design are aesthetic qualities and roadside, community, and environmental effects.

It has become apparent through operational experience and research that to successfully plan and design freeways and

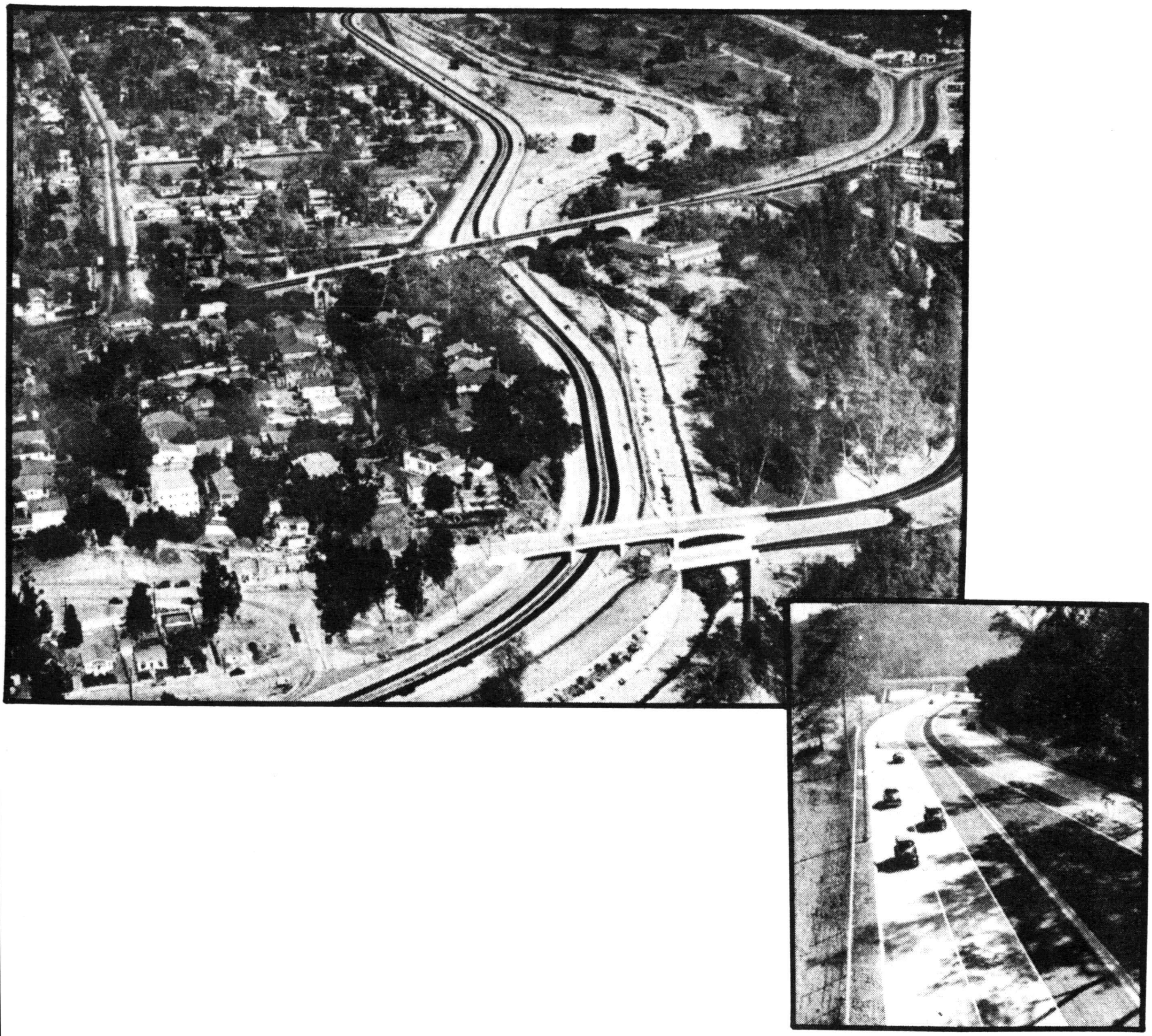

FIGURE 7 Arroyo Seco Parkway, Los Angeles.

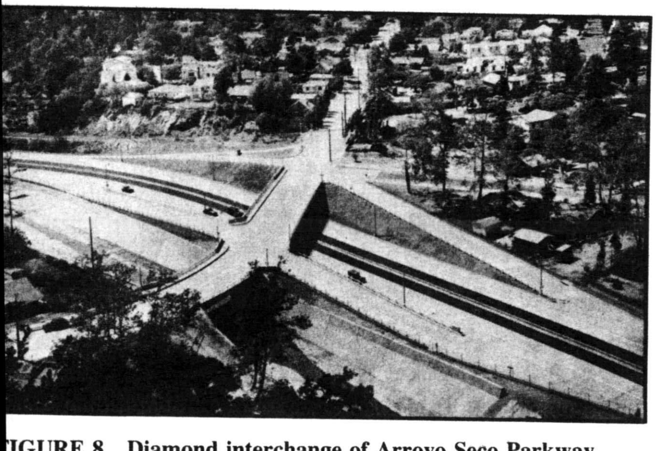

FIGURE 8 Diamond interchange of Arroyo Seco Parkway.

related facilities, certain basic concepts of design must be recognized and adhered to in practice. These same concepts apply in planning and design for freeway reconstruction. Consequently, this paper, after addressing the evolution of design concepts, will also present philosophical considerations in geometric design and the application of human factors in the planning and design process.

Freeway Design

Early freeways in urban areas were usually designed and constructed with design speeds of 50 to 60 mph. Today, 60 mph is the basic design speed considered. Many urban freeway reconstruction projects include upgrading a present design to 60 mph or greater as well as increasing capacity, modifying interchanges, and enhancing safety. In rural areas, 70 and 80 mph design speeds have become well accepted. In mountainous areas, a design speed of 60 mph has evolved as a basic criteria.

Cross Sections

The cross sections of freeways have changed rather dramatically since the 1930s. Although lane width criteria have changed little, the use of full and continuous shoulders, introduced in the 1950s, has become standard practice. This feature is particularly important on facilities with high-occupancy vehicle (HOV) treatments, to effect maintenance, and to efficiently and safely manage incidents. Such safety features as clear roadsides and flatter cut-and-fill slopes have evolved based on extensive research in the 1950s and 1960s. Safety appurtenances including the concrete barrier, safer guardrail and guardrail end design, crash attenuation devices, and frangible sign and lighting supports also were developed, making the highway environment a safer one.

Initially, freeways were constructed with two lanes in each direction of travel. The Arroyo Seco Parkway in Los Angeles, opened in 1941, was the first six-lane freeway. It was initially believed that six lanes (three in each direction) was the maximum that could be operated safely and efficiently. By the late 1950s, however, 8- and 10-lane freeways were being planned, designed, and built. Today, there are short sections (2 mi or less) of seven contiguous lanes on a number of freeways. Most of these in operation are basic 10-lane freeways (5 lanes in each direction) with 2 auxiliary lanes to accommodate entering, exiting, and weaving traffic between interchanges. In expanding a freeway beyond 10 basic lanes, consideration should be given to continuous collector-distributor roads. If frontage roads already exist, increasing the capacity of the corridor can be achieved through ramp metering, frontage road expansion, and frontage road and cross street signal coordination.

Medians

The separation of opposing lanes has progressively increased over the years. On early freeways, medians as narrow as 4 ft were prevalent. The 1945 Design Standards for the National System of Interstate Highways called for minimum medians of 4 ft in urban areas and 15 ft in rural areas; the 1956 Geometric Design Standards for the National System of Interstate and Defense Highways indicated minimum medians of 16 ft in urban areas and 36 ft in rural areas; the 1967 AASHTO publication *Highway Design and Operational Practices Related to Highway Safety* recommends a minimum median width of 60 to 80 ft. The latter dimensions are now being applied in rural areas. In some states, medians of 80 to 100 ft in open country are common on 70- and 80-mph freeways. In and near downtown areas, median widths of 24 to 30 ft, with inside shoulders and median barrier, are prevalent on new designs. Wider medians should be used on radial freeways in the intermediate and outlying areas of cities. Some metropolitan transportation studies recommend medians upwards of 70 ft in those locations to allow for future expansion—for added lanes, reversible roadway, HOV and exclusive bus lanes, or rail transit.

Sight Distance

Sight distance considerations along freeways and highways have changed and evolved over time. Although there have been some modifications in stopping sight distance, the new concept of decision sight distance (originally referred to as anticipatory sight distance) first surfaced in the late 1960s. It was initially quantified in 1970, recognized by AASHTO in 1984, and refined by AASHTO in 1990.

During many freeway and interchange reconstruction projects it was realized that horizontal stopping sight distance around curves was never provided. The location of concrete barriers, the construction of retaining walls in conjunction with lane additions, and more extensive use of directional or semidirectional ramps with tight geometry are just some situations in which the designer must be cognizant of potential horizontal stopping sight distance deficiencies.

Interchange Design

By the 1960s, it was realized that freeways and interchanges should be planned and designed as integrated systems because their geometric and operational characteristics are interrelated. If considered as such in the planning and design process a higher capacity, more operationally efficient and safer facility could be achieved. Much of this was realized through human factors research resulting in a greater understanding of the capabilities and requirements of drivers.

Interchange forms increased significantly in number between 1928 and the late 1950s—from the cloverleaf in Woodbridge, New Jersey, to the diamond in Los Angeles, to a broad array as depicted in the early AASHTO design policies of 1954 and 1957. By the advent of the Interstate system, interchanges were categorized into several basic types, including

- 3-leg (trumpet, directional T or Y),
- Diamond,
- Cloverleaf,
- Partial cloverleaf,
- Directional with loops, and
- All directional.

These types have not changed over the years; however, different geometric forms of these basic types have been developed and implemented and are part of the engineer's arsenal in providing freeway access. Two broad categories o interchanges related to the two interchanging facilities ar prevalent—service and system. Service refers to an inter change of the freeway with a surface street (i.e., arterial o collector street), and system refers to one between tw freeways.

Not only has there been a change in design approach in th specific geometrics of interchanges during the 1970s and 1980s but there has been a rethinking of the interchange selectio process. Much of this change has occurred as a result of better understanding of human factors and the realization tha consistency in operational characteristics along the freeway is more important than designing each individual interchange to precisely fit a future traffic forecast. The concepts of basic lanes, lane balance, single exit design in advance of the crossroad, all right-hand exits and entrances, minimum 1-mi spacing between interchanges, and no weaving within an interchange are now generally accepted practice. Interchange forms that are consistent with these operational guidelines are presently those that are considered for new locations or in modifying existing interchanges. Consequently, the diamond forms and some of the partial cloverleaf forms are prime candidates for service interchanges.

The system interchanges have evolved in a similar manner and for similar reasons as the service interchanges. In the 1950s the philosophy appeared to be to minimize the number of bridges in a system interchange. This produced some rather interesting designs with left- and right-hand exits and entrances, transposed mainline traveled ways, and weaving within the interchange between major turning and through traffic movements. One is shown in Figure 9. Fortunately, this interchange has been partially reconstructed and the operational and safety deficiencies eliminated. The basic forms of system interchanges usually considered today are the all-directional (Figure 10) or directional with one loop or two loops in opposite quadrants. The full cloverleaf should never be considered for a system interchange because of its excessive right-of-way requirement, potential weaving problems, and high accident experience.

There has been a continuing evolution in geometric design of freeways and interchanges since the 1920s. Much has been

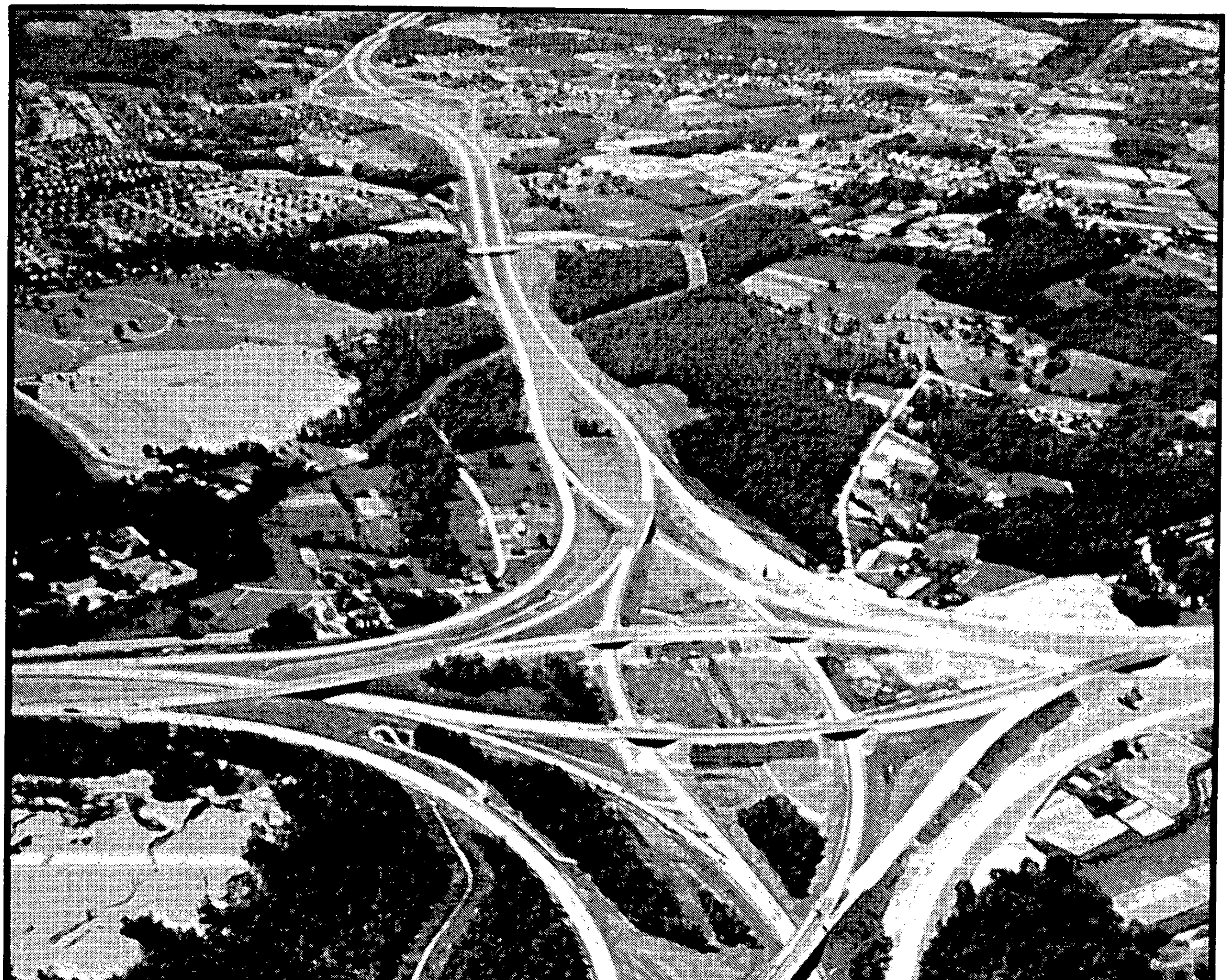

FIGURE 9 System interchange in Baltimore, Maryland (1950s).

FIGURE 10 All-directional interchange in California.

learned, yet there is still more to learn, to research, and to observe that can continue to assist in the design of safer and more operationally efficient freeway facilities. As we move forward into an era of greater use of freeway corridors for transit and HOV facilities, attention should be directed toward better integration of the modes. Also, with future implementation of intelligent vehicle highway systems, freeway facilities will continue to change and evolve to respond to different physical and human requirements.

REFERENCE

1. *A Policy on Geometric Design of Rural Highways.* AASHO Washington, D.C., 1954.

Publication of this paper sponsored by Committee on Geometri Design.

Interchange Planning and Design—An International Perspective

RUEDIGER LAMM, BASIL PSARIANOS, ELIAS M. CHOUEIRI, AND THEODOR MAILAENDER

Worldwide, interchange design issues represent one of the most important challenges in highway design. Existing norms, standards, or guidelines for interchange design in eight countries on three continents were reviewed, analyzed, and evaluated. Because of space and time constraints, only important issues of interchange planning considerations and principles, interchange types, design standards, local practices, and information on traffic safety are presented. It is interesting to note that for system interchanges, the Trumpet, Y and T interchange types, cloverleaf and directional interchanges are most commonly used, whereas for service interchanges, diamond and partial cloverleaf interchange types are commonly applied.

Significant traffic increases in the industrialized countries of the world have resulted in the need to increase capacity and efficiency and to improve traffic safety. In terms of safety, grade-separated intersections (interchanges), where capacity constraints are evident, and where the largest portion of accidents occur, represent some of the major deficiencies in highway design.

As a means to familiarize highway design engineers with national/international practices and experiences in geometric design, as related to safety and efficiency, the TRB Committee on Geometric Design nominated a committee of engineers to plan, coordinate, and conduct a session on interchange design issues during the 1992 TRB Annual Meeting.

The authors were consequently asked to collect, review, analyze, and evaluate existing interchange design guidelines, standards, or manuals to obtain an international perspective of current interchange design practices. To achieve the objective of this study, letters were sent to a number of major industrialized countries requesting copies of design guidelines and other information pertaining to interchange design.

Answers were received from Australia, Austria, Germany, Greece, Ireland, Norway, South Africa, and Switzerland. Information was also received from Italy, the Netherlands, and Sweden. Unfortunately, the information received from these three latter countries had to be excluded because the language used in the guidelines was not spoken or understood by the authors. Translations were not possible because of financial and time constraints. France, Great Britain, and Japan did not respond.

Results of this study, coupled with what is known in the United States, could serve as a basis for any future international investigations of interchange design issues, which could eventually help improve interchange design guidelines.

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INTERCHANGE PLANNING AND DESIGN IN AUSTRALIA

The information presented in this section is based on *Grade-Separated Interchanges: A Design Guide*, prepared by the National Association of Australian State Road Authorities (NAASRA), 1984. NAASRA has since become AustRoads, which is somewhat analogous to AASHTO.

An interchange, as it is defined in the NAASRA publication, is a combination of ramps and grade separations at the intersection or junction of two roads that is designed to eliminate at-grade turning and crossing movements on the major road carriageways. Interchanges on freeways are the only locations at which traffic can exit from or enter the through route. They should allow for the most efficient and convenient choice of route for the major traffic movements. Interchanges separate major crossing and turning movements and enable maximum traffic volumes to operate safely. Crossing conflicts are reduced or eliminated, and turning conflicts are minimized.

Interchange Planning Considerations

Justification for Providing Interchanges

Interchanges may be justified for any of the following reasons:

- To provide access to freeways,
- To increase capacity at critical intersections,
- To separate conflict points between movements with high relative speeds,
- To suit particular topography where an interchange can be built at a cost comparable to at-grade intersections, and
- To satisfy planning considerations for future traffic.

Location and Spacing

Interchanges are located in positions that provide convenient access to and from major traffic generators. Desirable minimum spacings between successive interchanges are 1.5 to 2.0 km in urban areas and 5.0 to 8.0 km in rural areas. Maximum spacings are less definite; however, a critical review should be made of traffic service provided by the total road system at spacings above 4.0 km in urban areas and 12.0 km in rural areas.

Lane Balance

Lane balance should be maintained at interchange exits and entrances by using auxiliary lanes if necessary. The three principles of lane balance are as follows:

- The number of lanes beyond the merge of two traffic streams should not be less than the sum of traffic lanes (or merging roadways) minus one, and not more than this sum.
- The number of lanes on the combined roadway before a diverge should be equal to or one less than the sum of all the traffic lanes following the diverge.
- The number of lanes on the through carriageway should be reduced by no more than one at any exit location.

However, the overriding consideration must be the achievement of a consistent level of service which will, among other things, ensure correct lane balance.

Evaluation of Alternatives

Study sketches of a range of likely layouts should be prepared after obtaining sufficient information on all factors likely to affect the choice of an interchange layout. These should be suitable to meet traffic needs and must be practical for the site and its controls. The various sketches are compared to select the most suitable layout, and preliminary designs and profiles are prepared. These preliminary designs are then analyzed with respect to design features, volume/capacity ratios and traffic performances, adaptability to possible changes, and estimated costs. A comparison of the alternatives will then enable the preferred layout to be determined.

Interchange Types

Interchanges fall into two main categories: those that join minor (secondary) road(s) with a freeway, and those that include only freeways (interstates, autobahnen).

The basic forms of interchange associated with minor roads include diamond [Figure 1(*a–f*)], partial cloverleaf (parclo) [Figure 1(*g–m*)], trumpet [Figure 1(*n*)], and bridged roundabout [Figure 1(*o*)].

The basic forms of freeway-type interchanges include the Y, T, cloverleaf, and directional types [Figure 1(*p–u*)]. Note that left-hand driving is the rule in Australia.

Interchange With Minor Road

Diamond Interchanges

The most common type of interchange with a minor (secondary) road in Australia is the diamond interchange. Because of its simplicity, it is the preferred interchange type for both urban and rural areas. The conventional diamond interchange has four ramps, which provide for all movements to and from the intersecting road. The three types of basic diamond interchanges are as follows:

- Conventional [Figure 1(*a–c*)]: Adaptable to a wide range of traffic volumes, with the ramp and minor road capacity only being limited by the ramp terminals. They require a single bridge and a bridge site clear of all ramps.
- Split [Figure 1(*d* and *e*)]: Suitable where capacity limitations of the minor road system or adjacent land use precludes the use of a conventional diamond. Capacity and efficiency are normally improved because traffic is distributed over two minor roads. The ramp terminals form a simple T junction, and acquisition along the minor road is minimized.
- Half diamond [Figure 1(*f*)]: Appropriate to situations in which traffic demand is predominantly oriented in one direction. The reverse minor traffic movements are accommodated by the minor road system, possibly leading to the next interchange.

Parclo Interchanges

Parclo interchanges are usually used when physical controls in one or more quadrants of the interchange preclude the use of a diamond interchange, especially in rural situations. An interchange with loops in advance of the minor road, with direction of travel on the freeway as the reference, is a Parclo A [Figure 1(*a–i*)]. An interchange with loops beyond the minor road is a Parclo B [Figure 1(*j–l*)].

In a Parclo A interchange, right turns from the minor road are made in advance of the grade separation structure, thus improving visibility and locating the queuing area outside the interchange. The provision of entry ramps from the minor road onto the freeway in the adjacent quadrants forms a Parclo A4 and minimizes the conflict on the minor road [Figure 1(*h*)].

In a Parclo B interchange, left turns from the minor road are made in the expected direction of travel, and wrong way movements are relatively difficult to make. Ramps in the adjacent quadrants of a Parclo B interchange form a Parclo B4 interchange [Figure 1(*k*)]. This increases the capacity of the interchange, but the double exit provision from the freeway complicates directional signing.

The provision of collector-distributor (C-D) roads with Parclo type A4 (and similarly B4) reduces problems associated with the two succeeding entry ramps onto the freeway and places the exit ramp nose in advance of the bridge [Figure 1(*i* and *l*)]. A combination of half of a Parclo A and half of a Parclo B interchange results in a Parclo A-B interchange, thus providing for all movements [Figure 1(*m*)].

Trumpet Interchanges

This type of interchange is suitable for three-way junctions. It is particularly suitable for connection of a freeway to a minor road and may also be suitable for some freeway-to-freeway interchanges [Figure 1(*n*)]. All movements are ac

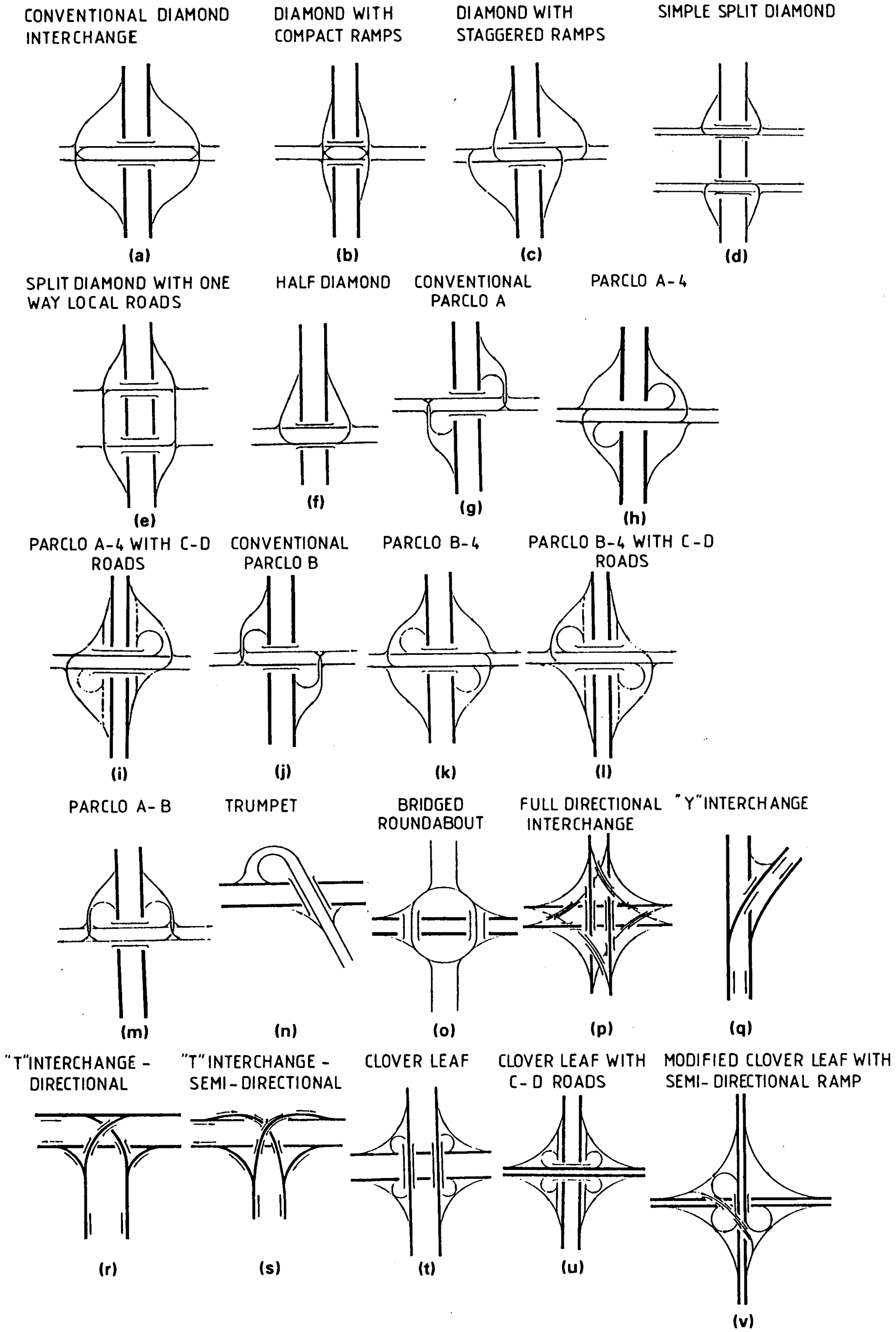

FIGURE 1 Interchange types in Australia.

commodated in this type of interchange, and the area of land required is relatively small, but long bridges are often necessary because of the angle of skew.

Bridged Roundabout

The bridged roundabout interchange is similar to the diamond interchange except that all turning movements are accommodated on the roundabout, which is separated from the through carriageways of the freeway. The complete interchange requires two bridge structures and four ramps. This type of interchange is particularly suitable for three or more intersecting roadways, excluding the freeway [Figure 1(*o*)]. All movements in the interchange are reduced to merges, diverges, or weaves, which facilitate ease of signing, and wrong way movements on exit ramps are less likely than at diamond or parclo interchanges.

Freeway-to-Freeway Interchanges

Directional Interchanges

A directional interchange is one in which direct ramps are provided for one or more right turn movements [Figure 1(*p*)]. A direct ramp would generally be used where the turning volume equals or exceeds the through volume. Where the right turning volume is less than the through volume, a semidirect ramp may be suitable.

Y and T Interchanges

An interchange for a three-way junction can take the form of a Y interchange with direct ramps [Figure 1(*q*)], or a T interchange with direct or semidirect ramps [Figure 1(*r* and *s*)]. The basic Y freeway-to-freeway interchange does not accommodate all movements, but is oriented to serve the major traffic demand in one particular quadrant. The T interchange accommodates all movements and, in the case of the directional interchange, the right-turning traffic to and from the leg of the T diverges and merges on the right side of the through traffic.

Cloverleaf Interchanges

The cloverleaf interchange provides indirect right turn movements in all four quadrants by means of loops. Cloverleaf interchanges are generally used where the turning and weaving volumes are relatively low [Figure 1(*t*–*v*)].

Interchange Design Standards

Tables 1 and 2 present important design parameters and speed relationships for Australian interchanges. Appropriate ranges of desirable ramp elements widths, grades on ramps, and typical design speeds for ramp tapers are also provided in the Australian Design Guidelines.

Sight Distance

The freeway alignment should desirably provide 200 to 300 m of stopping sight distance to an exit nose for high-volume, high-speed operations and up to 150 m where traffic volumes and operating speeds are relatively low. A sight distance of 200 m should be provided from 140 m before the nose to 60 m along the ramp. Where the minor road overpasses the major road, the sight distance to the ramp terminals should be consistent with the operating speed of the local through traffic. Exit ramp layouts should provide a sight distance of 120 m to the terminal traffic island to facilitate driver orientation.

Auxiliary Lane

An auxiliary lane is desirable when weaving distance between an entrance ramp and a following exit ramp is less than 600

TABLE 1 Recommended Design Speeds for Interchange Elements

Form of Interchange	Type of Connection	Range of Freeway Design Speeds (km/h) 100-120	80-100	60-80
		Range of Ramp design speeds (km/h)		
Freeway to Freeway	Loop	50-70	40-60	40-60
	Semi-Direct	70-90	60-80	50-70
	Direct	80-100	70-90	60-80
Freeway to Minor Road	Loop	40-60	40-60	40-60
	Semi-Direct	60-80	60-70	50-70
	Direct	60-90	60-80	60-80

TABLE 2 Range of Curve Parameters for Interchange Ramps

Design Speed (km/h)	Radius (m)	Superelevation (%)	Assumed Coefficient of Side Friction
110-130	650-800	8-5	0.12-0.11
90-100	300-450	10-5	0.18-0.12
70-80	100-150	10-6	0.31-0.26
40-60	40-75	12-6	0.35-0.33

m. It can also be placed between more widely spaced ramps if a capacity analysis (taking into account grades and the percentage of trucks) indicates that an additional lane is warranted.

Clearances

For all interchange roadway elements, an absolute minimum clearance of 1 m should be provided from the outer edge of the carriageway to any lateral obstructions. In no case should vertical clearances be less than 5.5 m.

INTERCHANGE PLANNING AND DESIGN IN AUSTRIA

The information presented in this section is based on *Intersections, Mixed and Grade-Separated, RVS 3.43*, prepared by Committee on Planning and Traffic of the Austrian Association for Engineers and Architects, 1991.

General Design Considerations

The general criteria applied in the Austrian Guidelines RVS 3.43 for the design of interchanges are as follows:

- Site selection should be based on appropriate topography, right of way, and sight distance;
- In terms of conceivability, the uniform design of the single intersection parts of mixed or grade-separated intersections is more important than the uniform design of the intersections;
- Sufficient temporal and spatial distances between decision points should be strived for;
- Exits should be placed before entrances.

Interchanges in the Austrian guidelines fall into two categories: mixed (i.e., interchanges with at-grade and grade-separated intersections) (Figure 2) and system (i.e., grade-separated intersections of all traffic forms) (Figure 2).

Mixed Interchanges

The basic forms of mixed interchanges in Austria include parclo [Figure 2(*a*–*d*)], diamond [Figure 2(*e*)], half-diamond and quarter-cloverleaf [Figure 2(*f* and *g*)], trumpet [Figure 2(*h* and *i*)], and one-quadrant [Figure 2(*j*–*m*)].

The Austrian guidelines include the following mixed interchange types:

- Parclo: The first type of parclo has the best advantage of providing outside left turn movements (B-C and D-A). The distance between at-grade intersections is well defined, with minimum right-of-way (ROW) and favorable corner connections [Figure 2(*a*)].
- Diamond: Diamond interchanges are simple and accommodate all traffic movements [Figure 2(*e*)]. Half-diamond interchanges are especially suitable for half-mixed interchanges.
- Half-diamond and quarter-cloverleaf: Depending on the quadrant in which the half-diamond and quarter-cloverleaf are set up, favorable connections can be provided [Figure 2(*f* and *g*)].
- Trumpet: Examples of trumpet interchanges are shown in Figure 2(*h* and *i*) with one at-grade intersection. The one shown in Figure 2(*h*) is less economical because one more bridge is needed.
- One-quadrant: One-quadrant interchanges [Figure 2(*j*–*m*)] are favorable if there is only one corner connection with high traffic volume. The application of a one-quadrant interchange may fit the topography well.

System Interchanges

The Austrian guidelines included show the following system interchange types:

- Full cloverleaf interchanges are the most economical solution with little ROW.
- Full cloverleaf interchanges with one or more semi-directional ramps eliminate weaving sections for high-volume corner connections [see Figure 3(*i* and *j*)].
- Other types of interchanges are also used for certain situations.

System Selection

For the selection of an interchange system, the following factors must be considered: road-network system, traffic movements, traffic volume, traffic safety, traffic capacity, environmental impacts, economy and costs, number of conflict points, conceivability and recognition, position of roadways to be connected, topography, ROW, fixed points, spacing, and signing.

Alignment and Cross-Section Design

The minimum lengths of transition curves, radii of vertical curves, superelevation rates, and minimum sight distances are based on the operating speed (V85). Operating speed is defined as the 85th-percentile speed of passenger cars under free-flow conditions for dry and clean road surfaces at a certain section of roadway. This speed depends on the horizontal alignment, expressed by the radii of curve (see Table 3), and on the vertical alignment, expressed by grades (see Table 4). Depending on the design in plan and profile, the smaller value of operating speed becomes decisive for the design. Minimum operating speeds for ramps are 40 km/hr for Interstates and freeways (30 km/hr for exceptional cases) and 30 km/hr for all other roads.

Horizontal and Vertical Alignment

1. A short tangent between circular curves in the same driving direction must be avoided.

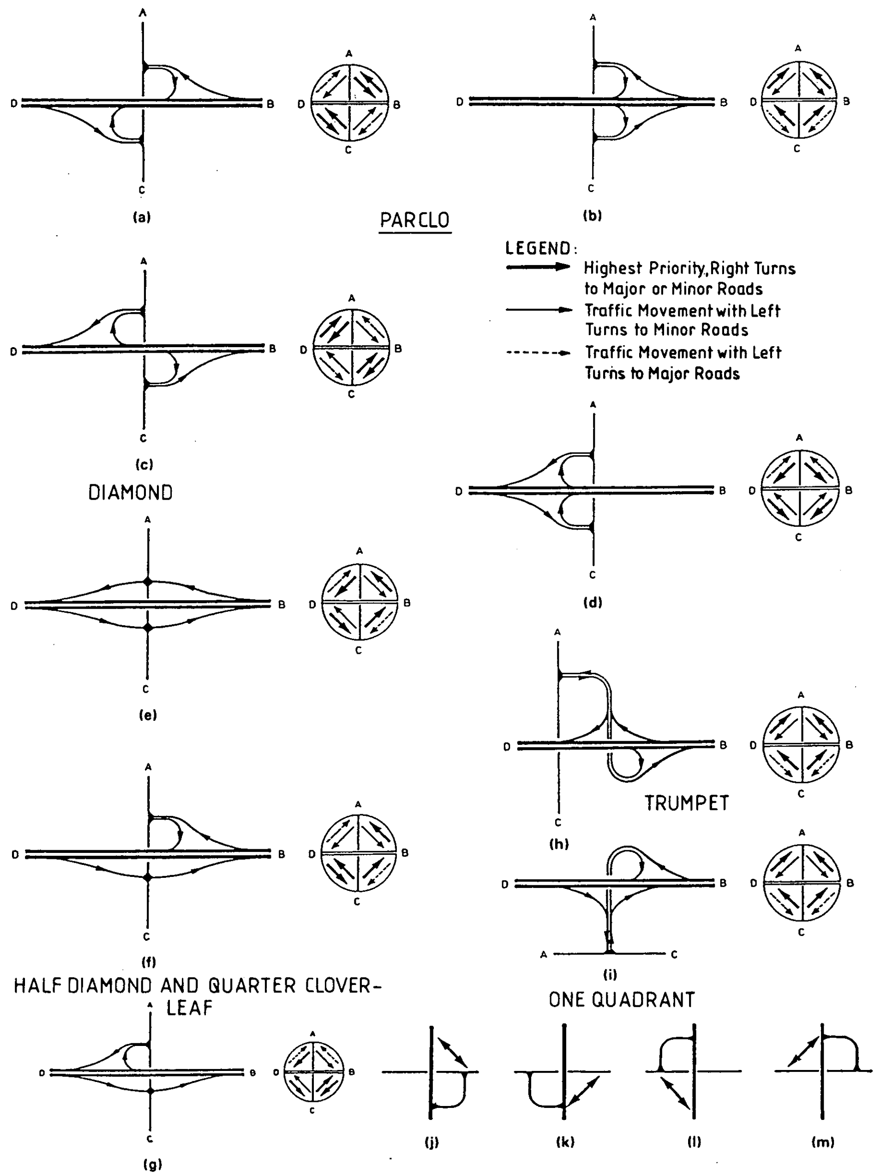

FIGURE 2 Mixed interchange types in Austria.

TABLE 3 Minimum Radii of Curve Relative to Operating Speeds

V85 (km/h)	30	40	50	60	70	80	100	130
R (m)	15	30	50	80	130	200	400	800

TABLE 4 Maximum Grades Relative to Operating Speeds

V85 (km/h)	80	90	100	110	120	130
S (%)	8	7	6	5	4.5	4

2. Because of ROW, radii as small as possible must be used
3. Minimum radii or circular curves are presented i Table 3.
4. The clothoid is considered a transition curve. Minimu lengths of clothoids are presented in Table 5. Maximum length of clothoids should not be more than double the minimu length ($L\max = 2\ L\min$).
5. Maximum grades of the vertical alignment are presente in Table 4. For upgrades, the maximum grade is limited to percent; for downgrades, the maximum grade is limited to percent.
6. Minimum radii of crest and sag vertical curves are show in Table 6.

TABLE 5 Minimum Lengths of Clothoids Relative to Operating Speeds

V85 (km/h)	30	40	50	60	70	80	90	100	110	120	130
Lmin (m)	10	15	20	30	39	44	50	56	61	67	72

TABLE 6 Minimum Radii for Crest and Sag Vertical Curves Relative to Operating Speeds

V85 (km/h)	30	40	50	60	70	80	90	100	110	120	130
Rmin (m) (Crest)	175	375	700	1200	2000	3000	4500	6500	9000	12000	15000
Rmin (m) (Sag)	200	400	650	900	1300	1700	2100	2600	3100	3700	4500

Cross Section

The pavement widths for one-lane ramps are presented below relative to radius of curve.

Radius R (m)	*Pavement Width (m)*
$R \geq 50$	4.0
$30 < R < 50$	4.0
$R \leq 30$	5.0 + widening

A paved shoulder of 0.5 m must always be provided. A divided two-lane ramp is justified when traffic volume exceeds 1,500 vehicles per hour and the percentage of trucks is about 15 percent.

The minimum superelevation rate is 2.5 percent. The maximum superelevation rate for ramps of mixed interchanges is 7 percent; for system interchanges, the maximum superelevation is 6 percent. Other values of superelevation rates that apply to specific situations are presented in the guidelines. They are not included here because of space constraints.

Sight Distance

Two types of sight distances, both related to operating speeds, are distinguished in the Austrian guidelines: stopping sight distance and operating sight distance. Values for these sight distances are presented in the guidelines. They are not presented here because of space constraints. Means of calculating sight distances, as in other guidelines, take into account perception and reaction times, as well as braking distance.

The operating sight distance is based on a new concept, and is presented for the first time in this study of international interchange design guidelines. The operating sight distance is defined as the distance required by a driver to stop his or her vehicle in front of a large, unexpected obstacle (usually a stopped vehicle) by an operating braking maneuver (i.e., emergency braking does not have to be used). Therefore, the factor of wet pavement for calculating stopping sight distance is not significant here to operating braking maneuvers. Operating sight distances are, therefore, longer than stopping sight distances.

INTERCHANGE PLANNING AND DESIGN IN GERMANY

The information presented in this section is based on *Guidelines for Highway Design of Rural Roads (RAL): Part III: Intersections (RAL-K), Section 2: Grade-Separated Intersections (RAL-K-2)*, prepared by the German Road and Transportation Research Association, 1976.

A grade-separated intersection (interchange) is a combination of several parts, consisting of through lanes, ramp exits and entrances, connecting ramps, and the like, which require various actions by drivers. Therefore, an important element of interchange design is the proper assembly of these single parts. In the German design guidelines, interchanges fall into two main categories: those that include only interstates (autobahnen), and those that join secondary road(s) with an interstate outside built-up areas.

Principles of Interchange Design

General Requirements

The following principles should be followed in the design and equipment of grade-separated intersections:

- A safe flow for all traffic movement should be provided.
- The capacity of single intersection parts should be well dimensioned in order to provide a smooth traffic flow with road sections outside the interchange.
- An interchange must be easily perceived and should be well designed to provide easy maneuvering by drivers (a safety criterion).

Safety

Distinct (easily recognized) and well-timed and well-placed signing are keys to safe and efficient operation, for informing, warning, and controlling drivers. This is especially important for drivers who are not familiar with the area. Exit and entrance areas have to be distinguished by the alignment itself, by vertical and horizontal guidance, and by signing devices. Distinctness is guaranteed if, for each intersection part, sufficient sight distances are provided, which would allow the driver to easily perceive upcoming design elements. In terms of conceivability, the proper design of the individual intersection parts of grade-separated intersections is important. The key is to strive for adequate temporal and spatial distances between decision points. Minimum design elements, such as breaks in the horizontal alignment, which would require operating speed reductions, must be clearly visible. In these cases, proper signing, including chevrons in some cases, should improve perception considerably. The "easy-to-follow" requirement for a grade-separated intersection is made available when sufficiently long transition sections are provided for necessary operating speed changes and when the design of loop to ramps satisfies sound driving dynamic considerations.

Capacity

The capacity of one through traffic lane is about 1,800 vehicles per hour. Exceeding this traffic volume could result in an inadequate quality of traffic flow in entrance, exit, and weaving sections.

Location and Spacing

The selection of through lanes in a grade-separated intersection depends on the location of the interchange, traffic volume, characteristics of intersecting roads, and turning movements. For interstate-to-interstate interchanges, the through lanes are assessed on the basis of the dominant traffic directions within the observed road network. For interchanges with secondary roads, the interstate (autobahn) is always the dominant road; through lanes follow the course of the interstate. Entrance and exit ramps should always be located in straight (tangent) sections. Allowable minimum spacings between successive interchanges are 1.1 to 1.6 km. Desirable minimum spacings have to be calculated from given equations. On average, minimum spacings are 3.2 to 3.7 km for interstate-to-interstate interchanges, 2.7 to 3.2 km for interchanges with minor roads (high traffic volume), and 2.2 to 2.7 km for interchanges with secondary roads (low traffic volume).

Interchange Types

The basic forms of interstate-to-interstate interchanges in Germany include trumpet, T and Y, cloverleaf, and directional. The basic forms of interchanges associated with minor roads include parclo and diamond. For grade-separated intersections, three types of ramps are used, depending on traffic movements: direct [Figure 3(*a*)], semidirect [Figure 3(*b*)], and indirect [Figure 3(*c*)].

Interstate-to-Interstate (Autobahn) Interchanges

Related to interstates (autobahnen), several different forms of three- and four-leg interchanges are used in Germany.

Trumpet

The Trumpet Interchange has the following characteristics [Figure 3(*d*)]:

- Suitable for three-way junctions,
- Limited ROW,
- Economical solution,
- Small radii and low operating speeds in ramps, and
- Accommodates all traffic movements.

T and Y Interchanges

T interchanges have the following characteristics [Figure 3(*e* and *f*)]:

- Suitable for three-way junctions,
- Semidirectional turning movements,
- Higher ROW and costs than those of trumpet interchanges,
- Generous ramp design,
- Design speed up to 50 mph (80 km/hr), and
- Caters to all traffic movements.

Y interchanges [Figure 3(*g*)] serve major traffic demand in one particular quadrant.

Cloverleaf Interchanges

All left-turn traffic movements in the cloverleaf interchange are carried out by means of loops. This is the most economical type of freeway-to-freeway interchange as it requires only one bridge structure, a compact loop, and tangent ramps. This type of interchange is the most widely used four-leg interstate interchange in Germany. Unlike the United States, the Germans have had good experiences with cloverleaf interchanges.

The characteristics of cloverleaf interchanges are as follows:

- Suitable for four-leg intersection,
- Limited ROW,
- Economical solution,
- Low to moderate capacity,
- Collector-distributor lanes may be incorporated for ca pacity reasons, and
- Accommodates all traffic movements.

The speed limit for through lanes should not exceed 80 t 100 km/hr. The weaving lane should be at least 300 m lon (distance between exit and entrance ramp noses). In referenc to the four-quadrant interchange in Figure 3(*h*), different va rieties of designs of loops, tangent ramps, and distributor lane are introduced in Germany, depending on terrain, ROW location, access, network, traffic requirements, and the like Modified cloverleaf interchanges with semidirectional ramp for left turn movements of high traffic volumes are shown i Figure 3(*i* and *j*).

Directional Interchanges

Semidirectional interchanges [Figure 3(*k*)] and full-direction interchanges [Figure 3(*l*)] are applied when the capacity c cloverleaf interchanges, because of loop ramp design and lin ited lengths of weaving lanes, is no longer sufficient. In the

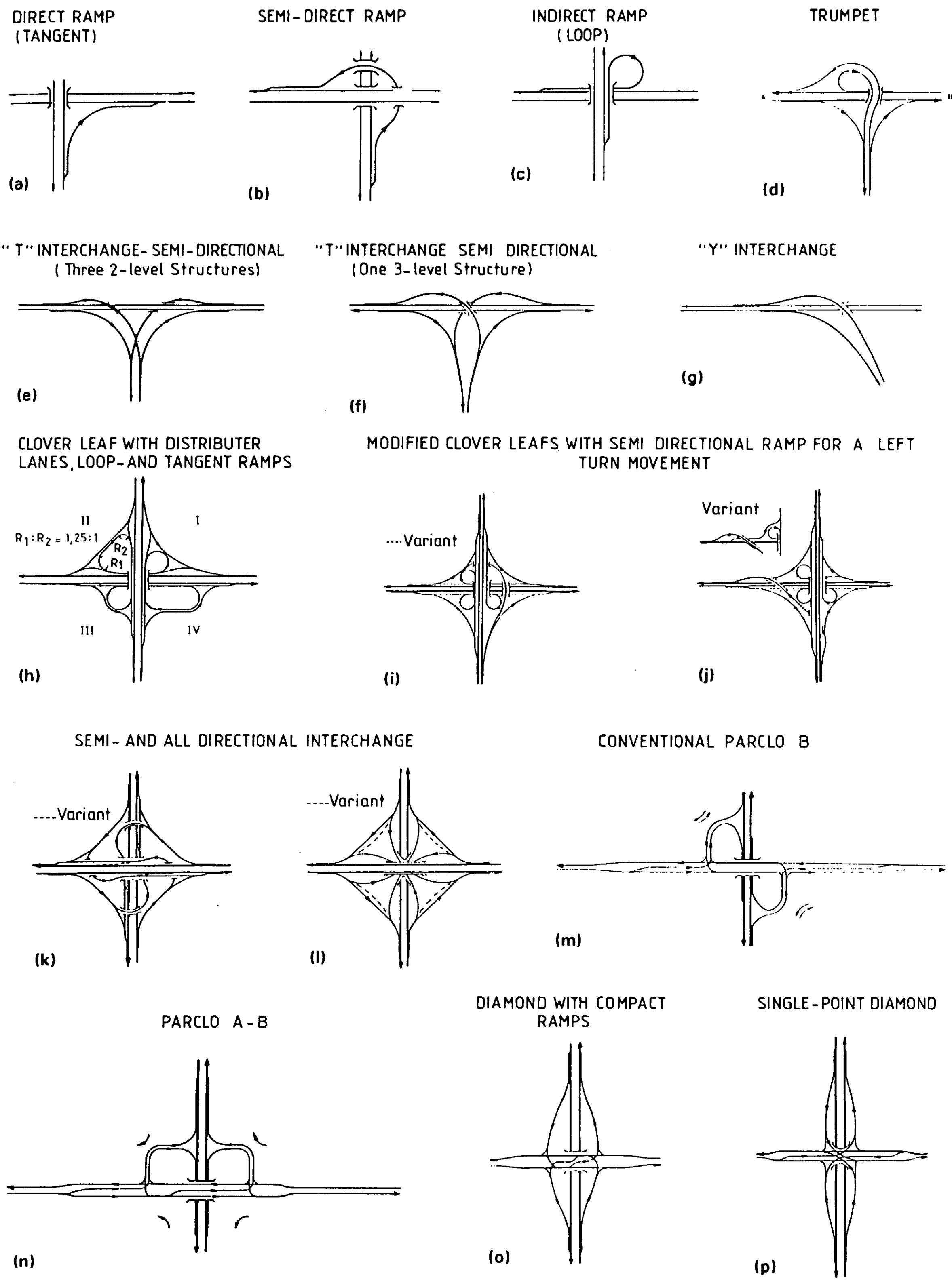

FIGURE 3 Interchange types in Germany.

cases, all or several left turn traffic movements are improved by semidirectional or full-directional ramps.

Interchanges With Secondary Roads

Parclo

In the German guidelines, parclo interchanges are applied in the same manner as, for example, in Australia and the United States. Therefore, this type of interchange is not discussed further here. Figure 3(m) shows, for example, the most commonly used Parclo B form. Other varieties in use are shown in Figure 1. A Parclo A-B type is shown in Figure 3(n).

Diamond

A diamond interchange has the particular advantage that it can be located within a relatively narrow land area because it needs little extra width beyond that required for the major road itself. Because of its simplicity, a diamond interchange is especially preferred in urban areas. Diamond interchanges usually require signalization where cross streets are carrying moderate-to-large traffic volumes. Often, clear (conceivable) signing is difficult to provide; consequently, wrong-way entry is possible, especially on the at-grade parts of the intersection. Typical diamond interchanges are shown in Figure 3(o and p). Numerous other configurations are in use (see Figure 1).

Interchange Design Standards

The road characteristic of through lanes in an interchange should be the same as that for those outside the interchange. Therefore, large differences in design speeds are not desirable. These are allowed only in exceptional cases when, for example, high traffic volumes in entrance, exit, or weaving sections may affect safety, capacity, and traffic flow. Only in these cases must speed limits be posted. Related to connecting ramps are two groups of ramps (Figure 4): connection between interstates (grade-separated to grade-separated) and connection of an interstate with a minor (secondary) road (grade-separated to at-grade). Various types of ramps are shown in Figure 4. Also in the figure are recommended speeds for unadjusted and adjusted ramps. Unadjusted means that local or spatial design constraints are nonexistent. Limiting values for design parameters, with respect to design speeds, are presented in Table 7.

INTERCHANGE PLANNING AND DESIGN IN SOUTH AFRICA

The information presented in this section is based on the *Manual for the Preparation of Detailed Geometric Design Plans for National Roads-G2*, prepared by the South African Roads Board (former the National Transport Commission), February 1984.

The Geometric Planning Manual defines the geometric design standards applicable to all declared national roads in South Africa. It is not intended to be a legal document, and it is published mainly for the information and guidance of the officials of the Department of Transport, the Provincial Administration, and for consulting engineers engaged in the planning and design of national roads. The policies on highway design practice published by AASHTO, as amended and updated from time to time, are approved references for use in conjunction with the G2 manual. Where standards differ, the standards in the G2 manual apply.

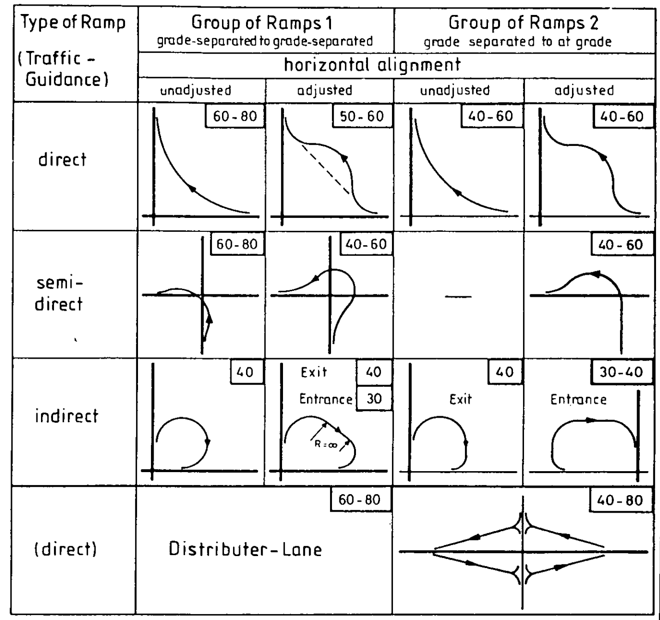

FIGURE 4 Types of ramps and recommended speeds (km/hr).

Interchange Planning Considerations

Types

Selection of interchange type is affected by many factors, including topography, proximity of other interchanges, speed, traffic, local street system and cost. Interchange types are classified as local street interchanges or access interchanges that are designed for stop conditions on one or more turning movements and freeway-to-freeway interchanges or major interchanges that are designed for free-flow conditions on all turning movements.

Spacing and Signposting

To ensure a high level of service for the foreseeable future the spacing between interchanges on national freeways shall be 8 km or more. Any closer spacing requires the approval of the director of the Division of National Roads. All interchanges must be properly signposted in accordance with the South African Road Traffic Signs Manual.

Lane Balance and Width

Design traffic volumes and capacities will determine the number of lanes required. This number may be increased or de

TABLE 7 Limiting Values for Design Parameters

Design Parameter		Design Speed V_d [km/h] 30	40	50	60	70	80
Minimum Radius of Curve	R [m]	25	50	80	130	190	280
Maximum Upgrade	+ s [%]	5,0					
Maximum Downgrade	- s [%]	6,0					
Minimum Crest Vertical Radius	R_c [m]	500	1000	1500	2000	2800	4000
Minimum Sag Vertical Radius	R_s [m]	250	500	750	1000	1400	2000
Minimum Superelevation Rate	e_{min} [%]	2,5					
Maximum Superelevation Rate	e_{max} [%]	6,0					
Minimum Stopping Sight Distance	SSD [m]	25	30	40	60	85	115

creased in some sections to ease operation. The following lane balance principles should be followed:

- The basic number of lanes beyond the merging area should be the sum of all merging traffic lanes minus one.
- The basic number of lanes before the diverging area should be the sum of all diverging traffic lanes minus one.
- No carriageway may be reduced by more than one lane at a time.
- All traffic lanes including auxiliary lanes must be 3.7 m wide.

Design Speed of Crossroad

n rural areas, the design speed of the existing crossroad must e used. This should preferably not be less than 100 km/hr. n suburban areas, the design speed of the crossroad must be aken as 10 km/hr in excess of the existing speed limit on that rossroad or 80 km/hr, whichever is greater. Control of access nust be enforced on the crossroad at least 160 m beyond the amp intersections.

amp Design

- Off ramps at access interchanges must be long enough or nust be widened to provide sufficient storage for vehicles so hat interference with through traffic is avoided.
- A traffic lane width of 4 m is specified for ramps at terchanges.
- The maximum grade on ramps should be 8 percent, but percent is preferred. A 3 percent maximum grade to the mp intersection shall be used.
- Ramp intersections must be placed to meet the crossroad where the grade on the latter does not exceed 3 percent.
- The design speeds of ramps depend on the design speed of the national road and should not be less than those specified below:

National Roads (km/hr)	*Ramp (km/hr)*
60	30
80	40
100 and above	50

- In general, a design speed of 40 km/hr is considered desirable for loops.
- The design speeds for directional ramps depend on the design speed of the national road and should not be less than those specified below:

National Roads (km/hr)	*Ramp (km/hr)*
60	55
80	65
100	75
120	85
140	95

- Diamond ramps should be designed for a speed of at least 80 km/h in the vicinity of the nose, but only 30 km/h to 40 km/h near the intersection with the cross-road.
- The minimum radii for the reference lines of ramps are specified below:

Design Speed (km/hr)	*Minimum Radius (m)*
20	30
40	45
50	70
60	110
70	160
80	210

Notes on Local Practice in South Africa

- Diamonds, because they are the type expected by motorists, are preferred for access interchanges. Parclo interchanges can be used, but generally only where physical conditions so dictate; Parclo A interchanges are preferred.
- "Crossroad over" is preferred because of assistance with deceleration and acceleration and better visibility.
- Quadrants are landscaped to make accidental departures from ramps safer and to eliminate guardrail for sight distance reasons.
- Diamonds don't include left signals until traffic dictates addition of signals.
- A number of full cloverleaf interchanges exist as system interchanges, but new interchanges of this type will not be built because of the weave problems they generate. A loop in the minor flowing direction would be used today. However, few system interchanges are built in the current funding situation.
- Only one double diamond (3-level diamond) exists in a location where space was restricted.
- No single-point diamonds have been built, but one is being considered as an additional interchange on the existing freeway between Johannesburg and Pretoria.

INTERCHANGE PLANNING AND DESIGN IN SWITZERLAND

The information presented in this section is based on *Intersections, Part SNV 640263, Principles; Part SNV 640265, Types; Part SNV 640266, Elements*, prepared by the Swiss Association of Road Specialists, 1972.

The planning and design of interchanges in Switzerland is based on an extensive amount of information on the following factors:

- Geographical, physical, and political data [e.g., geology, topography, border lines (state, county, etc.)];
- Land use (e.g., zoning, existing facilities, utilities, structural lines, access control, population data);
- Traffic (e.g., traffic analysis, existing road network, mass transportation system, pedestrian facilities);
- Existing projects (e.g., general access projects, projects for the road network); and
- Funding (e.g., existing funds from federal, state, county, municipality, land acquisition, construction, maintenance and operation costs).

Types and Classification

An interchange is characterized by connecting road types (freeways, arterials, connectors, collectors, local roads), number and type of lanes per leg, permissible traffic movements, number of levels, urban or rural area, type of traffic regulation, pedestrian and bicycle traffic, and operation of mass transportation.

The type of intersection depends on the type of connecting roads. Types of roads that connect intersections are shown in Figure 5. As the figure shows, there are eight intersection types: Types 1, 2, and 3 for at-grade intersections; 4, 5, and 6 for at-grade or grade-separated intersections; and 7 and 8 for grade-separated intersections. Type 8 can be designed for more than two-level structures. Type 4 is avoided. For grade-separated intersections, the traffic lane arrangements for Types 7 and 8 are proposed in the Swiss guidelines (Figures 6 and 7).

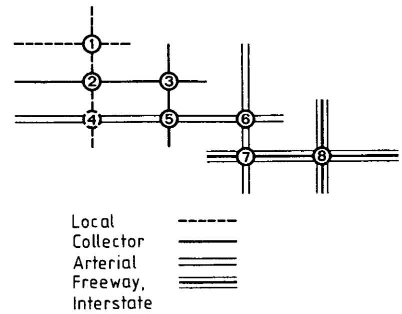

FIGURE 5 Numbering of intersection types with regard to connecting road types in Switzerland.

Important operational features of intersection Types 5–8 are presented in Table 8. The information is limited to design speeds, number of lanes, acceleration and deceleration lanes, and entering traffic volume, as related to various intersection and road types.

Lane Widths

- The lane width depends on the type of lane, such as through or ramp lane, type of road, and location of the intersection/interchange.

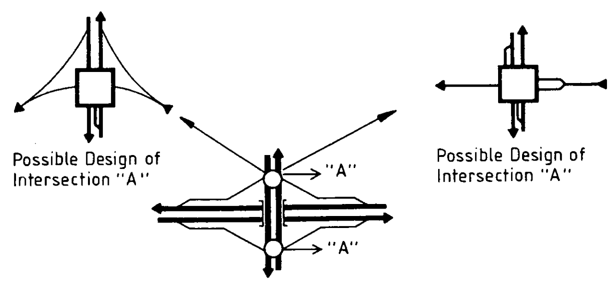

FIGURE 6 Traffic lane arrangement for interchange Type 7.

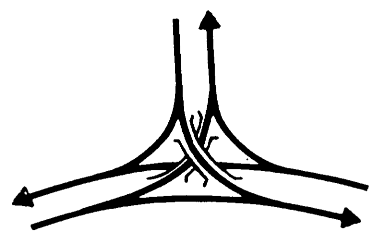

FIGURE 7 Traffic lane arrangement for interchange Type 8.

• As a general rule, and unlike open roadway sections, the through lanes within intersection areas should remain unchanged.

• Entrance and exit lanes, as well as acceleration and deceleration lanes, can be narrow.

• In curved sections, the principles of road-widening apply, as in the case of normal roadway curves.

• When bicycle volume exceeds 100 bicycles per hour, the lane widths within the intersection area should be widened by as much as 0.5 to 1.0 m, or extra bicycle lanes should be added.

• The lane widths for through lanes and ramp lanes, with or without shoulders, in intersection/interchange areas are presented in Table 9.

INTERCHANGE PLANNING AND DESIGN IN OTHER EUROPEAN COUNTRIES

Greece

Greece is a typical country in which official technical norms, standards, or guidelines for intersection and interchange design do not exist; work is currently under way to formulate guidelines in the near future.

Intersections and interchanges are planned, designed, and constructed on the basis of AASHTO guidelines (*A Policy on Geometric Design of Highways and Streets*) and the German Guidelines. The following interchange types are often used:

- Diamond (most favorable),
- Parclo,
- Trumpet,
- Cloverleaf, and
- Modified and mixture of the above-mentioned types.

As is the case in France and Italy, interchanges in Greece have tollbooth stations. As a result, provisions must be made on entrance ramps. In turn, certain modifications and mixtures of the different types of interchanges discussed in this study are often used.

Ireland

Interchange design standards in Ireland generally follow the procedures adopted in the United Kingdom and detailed in the United Kingdom Department of Transport Standard TD 22/86. A number of changes are made to suit traffic levels and driver behavior in Ireland.

TABLE 8 Design Criteria for Intersection Types 5-8 in Switzerland

Intersection Type	5		6		7		8	
Road Type Design	SS	HVS	HVS	HVS	HVS	HLS	HLS	HLS
Speed (km/h)	50-80	60-120	60-120		60-120	80-120	80-120	
No. of Lanes	2	2/4	2/4		2/4	≥ 4	≥ 4	
Acceleration/ Dec. Lanes						X	X	
Entering Traffic Volume (pcu/h)	500-3500		1000-4000		> 1500		> 2000	

Legend: SS = Collector; HVS = Arterial; HLS = Freeway/Interstate; and PCU/H = Passenger Car Units per Hour.

TABLE 9 Lane Widths in Intersection and Interchange Areas

Lanes	Freeway		Arterials		Collectors		Local	
	ST	MIN	ST	MIN	ST	MIN	ST	MIN
Through Lanes	4.00	3.50	3.75	3.00	3.50	3.00	3.00	2.75
Ramp Lane WTPS	5.00	4.50	5.00	4.50	5.00	4.50	4.00	4.00
Ramp Lane WPS	4.00	3.50	5.00	4.50	5.00	4.50	4.00	4.00
Paved Shoulder	2.50	2.50	NO PAVED SHOULDERS REQUIRED					

NOTE: Lane widths are in meters.
Legend: ST = Standard; MIN = Minimum; WTPS = Without Paved Shoulder; and WPS = With Paved Shoulder.

Interchanges are provided at all junctions on motorways and at major junctions on certain nonmotorway roads. They are generally either grade-separated roundabouts or grade-separated diamond types (possibly with small roundabouts at the ramp/minor road intersections). The actual layout chosen is determined by taking into account costs, topography, and traffic flows.

General Design Parameters

The design speed for the motorway is 120 km/hr. For connecting ramps, it is 60 to 80 km/hr, and it varies with size for roundabouts.

The minimum radius for normal roads is as follows:

$$R = \{V^2/[127(s + f)]\}$$

where

R = minimum radius (m),
V = design speed (km/hr),
s = superelevation rate (m/m) (max = 0.07), and
f = side friction factor (0.15 for 60 km/hr, 0.14 for 80 km/hr.

The minimum sight distance for normal roads is as follows:

$$SD = 0.56\ V + [V^2/(254\ cf)]$$

where

SD = sight distance (m),
V = design speed (km/hr), and
cf = coefficient of friction (0.34 for 60 km/hr, 0.32 for 80 km/hr).

The minimum longitudinal gradient for normal roads is 0.3 percent with curbs; no minimum applies if there are no curbs.

The maximum longitudinal gradient is 4 percent if the ramp carries a large volume of heavy trucks, up to 8 percent if a grade assists a change in speed, and up to 10 percent on minor, low-volume ramps.

The cross-section width on one-lane ramps is 1.5 m for the left shoulder, 4.0 m for the carriageway, and 0.5 m for the right shoulder. On two-lane ramps, the width is 1.5 m for the left shoulder, 7.5 m for the carriageway, and 0.5 m for the right shoulder.

Norway

Selection of Intersection Type

Grade-separated intersections (interchanges) should be considered outside urban areas when annual average daily traffic (AADT) exceeds 10,000. On some national routes, the limit is 5,000 AADT. Intersections on motorways shall always be grade-separated. Intersections on roads reserved for motor vehicles shall normally be grade-separated. In general, these are two-lane roads with two-way traffic (called Motorway B, even though there is only one lane in each direction).

Three types of interchanges are recommended: trumpet, diamond, and parclo. Mixtures of these types may also be used. To avoid conflicts between vehicles and pedestrians and cyclists along the secondary road, a parclo intersection with both ramps on the same side of the secondary road may be used.

Design

Today, it is essential to have auxiliary lanes of high standard in Norway. Deceleration lanes should always be used. When the speed limit of the primary road is 70 km/hr or higher, parallel deceleration lanes are recommended. Normally, acceleration lanes should be built. An acceleration lane should always exist and be of the same width as the through traffic lane (but not more than 3.5 m), with a short taper at the end. In Norway, this traffic situation is regulated with merging signs so that vehicles on the acceleration lane have the same right as the vehicles on the through traffic lane. However, the driver of the accelerating vehicle should adjust the speed to the speed level of the through traffic. With high standard on the auxiliary lanes, the standard of the ramps can be moderate.

Most Norwegian interchanges have at-grade intersections between ramps and the secondary road. Traditionally, these parts of the interchanges have the highest accident rates and can also cause capacity problems. The new guidelines recommend the use of roundabouts as a standard solution in the connection between ramps and secondary road on each side of the primary road. Roundabouts are safe; they provide high capacity and are often flexible for variations in traffic. It is interesting to note that even guidelines for interchanges in tunnels exist in Norway.

SUMMARY

To achieve the objective of this study, the authors reviewed, analyzed, and evaluated norms, standards, and guidelines for interchange design in Australia, Austria, Germany, South Africa, Switzerland, and others. For Greece, Ireland, and Norway, only abbreviated versions were discussed here to avoid repetition. For the countries under study, the following topics were investigated: (a) interchange planning considerations or principles of interchange design, including information on location and spacing, lane balance, safety, capacity and the like and (b) interchange types, such as freeway-to-freeway interchanges (often called system interchanges) and freeway-to-minor road interchanges (often called mixed or service interchanges).

As a result of this study, the following conclusions can be drawn. With respect to system interchanges (see Figures 1–3), the types of three-leg interchanges given high priority in nearly all the countries under study were the trumpet and Y and T types.

The four-leg interchanges given high priority were full cloverleaf, cloverleaf with semidirectional ramps, and semi- and full-directional types. It should be noted that, contrary to U.S. experiences, all types of cloverleaf interchanges are considered favorable solutions, as long as the weaving traffic volumes are not extremely high.

The following types of mixed interchanges were in use: conventional-, split-, single-point-, and half-diamonds; Parclo A, Parclo B, and Parclo A-B; bridged roundabout; one quadrant; and modified types, such as half-diamond and quarter-cloverleaf.

The following also were found in the study: design standards, including design and operating speed, horizontal and vertical alignment, and cross-section design for ramps; information on local practice; and information on traffic safety.

Because of space and time constraints, information on entrance, exit, acceleration, and deceleration ramps, as well as information on weaving lanes, sight distance considerations, equipment, signing, and the like, was not discussed in detail here.

The variability of approaches to interchange design in the countries under study, along with the experiences gained in the United States, should provide a basis for any future attempts to improve and extend existing norms, standards, and guidelines in the various countries of the world. In closing, future research on interchange design should take into account the differences in design philosophies as well as similarities between the countries under study as well as other countries, such as France, Great Britain, and Japan.

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Human Factors Associated with Interchange Design Features

HAROLD LUNENFELD

Interchanges are freeway design features that pose considerable safety and operational problems for drivers. Most freeway accidents and directional uncertainty occur in the vicinity of interchanges. Reasons for this phenomenon include friction between entering and exiting traffic, variability in design, and a high probability for driver error, the primary contributing factor of accidents. An introduction of general human factors considerations in highway design is presented; it is intended for highway engineers unfamiliar with the science of human factors. Examined are human factors issues associated with interchange design features, including driving task performance, driver error sources, reception of visually displayed information, information handling, driver attributes, sight distance, driver expectancy violations, and information presentation techniques and principles. Key human factors considerations in intersection design and operations are discussed.

Modern freeway design has resulted in safe and efficient travel that is unprecedented. A prime example is the Interstate system. Of the more than 4 million mi of highways in the United States, 45,000 are Interstate highway miles. These freeways carry approximately 22 percent of the more than 1.4 trillion annual vehicle miles traveled with a fatality rate of 1.03 per 100 million vehicle-mi (*1*). Problems on freeways are most likely to occur at interchanges (*2*). Among the reasons for this are (a) the friction that occurs at interchanges between entering and exiting traffic, (b) the high task demand of interchanges, and (c) the greater likelihood of driver error, the primary contributing factor of accidents. The purpose of this paper is to examine human factors issues associated with interchange design features.

HUMAN FACTORS CONSIDERATIONS

The human factors discipline links engineering (e.g., civil and traffic engineering) with behavioral science (e.g., psychology and physiology). It enables engineers to design and operate systems that are compatible with user characteristics, abilities, and limitations. In highway transportation, human factors help to determine driver characteristics and interfaces and gauge the effects of their interaction with traffic, the roadway, its information system, and the environment. Ultimately, human factors considerations are used to develop driver performance-related design and operational standards.

Federal Highway Administration, Office of Traffic Operations and Intelligent Vehicle-Highway Systems, U.S. Department of Transportation, 400 Seventh Street, S.W., Washington, D.C. 20590.

An appreciation of interchange-related human factors considerations is essential for design, operations, and safety. Engineers should have an understanding of drivers, their characteristics, and their performance. This will help them to optimize interchange designs, information displays, and operational procedures that contribute to driver error. It will enable them to match user characteristics with systems requirements and will allow them to accommodate a diverse and aging population.

DRIVING TASK PERFORMANCE

Human factors considerations of interchange design features should be viewed in the context of the driving task, which consists of three broad levels: control, guidance, and navigation. The complexity of information handling increases with each level. Control involves the driver's interaction with the vehicle and its controls and displays. Guidance involves the maintenance of a safe speed and path relative to the roadway and traffic. Pretrip navigation includes trip planning and route selection; in-trip navigation includes route following and direction finding (*3*).

Proper driving task performance generally results in the safe and efficient negotiation of the interchange, whereas driver error often leads to accidents and inefficient operations such as erratic maneuvers and missed exits. At interchanges, navigation tasks require directional decisions pertaining to the selection of paths to take to follow a route contained in a trip plan or selected in transit. Guidance tasks are predicated on the navigational decision (i.e., to take the interchange or stay on the mainline). They include speed and path decisions in response to the geometric design, performance of the requisite maneuvers safely and efficiently, and maintenance of an area of safe travel with vehicles in the traffic stream. Of primary importance is that drivers must perform guidance and navigation tasks in close proximity, which increases the chances of drivers becoming overloaded or committing errors (*4*).

The key to successful task performance at interchanges is efficient visual information gathering and error-free processing. Given that drivers have to perform multiple tasks often under extreme time pressures, information needs across all task levels must be satisfied. These needs should be satisfied through designs that do not violate driver expectancies or require demanding or unusual maneuvers and through traffic control devices that provide information when needed, where it can be used, and in a form most suitable to the demands of the situation (*5*).

DRIVER ERROR

Driver error is the principal contributing factor in most accidents and many instances of inefficient traffic operations. Errors can occur for a variety of reasons associated with driver physical or mental states and disabilities, task demands, and deficiencies in the design or operation of the highway (*6–8*).

Some driver errors are caused by alcohol or drug use, fatigue, inattention, lack of training or skills, lack of literacy, and such innate sensory-motor deficiencies as poor vision or hearing. There is little that engineers can do to eliminate those problems, which relate to licensing, education and training, and enforcement. In cases in which errors are committed because of the nature of the task, the demands of the situation, lack of visibility of the interchange, expectancy violations, or deficiencies in information display, the error sources can be eliminated. The information display can be enhanced, and the safety and operations of the interchange can be improved. Engineers can ameliorate the following interchange error sources:

- Excessive task demands;
- Unusual maneuvers or task requirements;
- Poor forward sight distance;
- Expectancy violations;
- Too much processing demand;
- Too little processing demand;
- Deficient, ambiguous, confusing, or missing displays;
- Misplaced, blocked, or obscured displays; and
- Small, illegible, or inconspicuous displays.

RECEPTION OF VISUALLY DISPLAYED INFORMATION

Vision is the most important sensory input channel. More than 90 percent of all information is received visually (*9*). A discussion of all pertinent visual reception factors associated with the receipt of information at an interchange is beyond the scope of this paper. However, the following important vision-related considerations should be taken into account in interchange design and operations.

Drivers must have the capability (e.g., visual acuity, color vision, etc.) to receive the information. The visual channel is selective. For interchange information to be received, it must be looked at and attended to. The interchange and its information displays must be within the driver's field of view and cone of clear vision at the freeway's operational speed.

Displayed information must be properly located. There must be sufficient sight distance. This includes both the sight distance to the interchange's design and its associated information treatments (signs, marking, delineation, etc.). The interchange information treatment must possess the physical characteristics (e.g., brightness, color, size, shape, contrast) necessary to be received and used in sufficient time to perform the requisite maneuver(s) safely and efficiently.

INFORMATION HANDLING

Driving is an information-decision-action task. Drivers gather information from sources internal and external to the vehicle and use it with information they bring to the task (knowledge, skills, expectancies, trip plans) to make decisions and perform control actions. In transit, drivers often have to do many things at one time. They generally have overlapping information needs associated with various activities. To satisfy these needs, they search the environment, detect information, receive and process it, make decisions, and perform actions using continuous feedback (*10*).

People are serial information processors in that they can handle only one source of visual information at a time. However, while negotiating an interchange, they often have to process a number of pieces of information and perform several activities at the same time. To do this, drivers juggle information sources and driving activities. They integrate activities and maintain an appreciation of a dynamic, constantly changing environment by sampling information in short glances and shifting attention from source to source. They rely on judgment, estimation, prediction, and memory to fill in the gaps, share tasks, and shed less important information and activities (*11*).

Memory

Drivers constantly handle information, with relevant sources transferred to short-term storage for rapid access, retrieval, and processing. The short-term memory trace lasts from 30 sec to several minutes, with a span of approximately 5 to 9 information sources (*4*). Information in short-term memory that is not relevant, reinforced, repeated, or retrieved and processed is forgotten. Important information may be transferred to the long-term memory, which has no limitations on the amount of information it can store, or on the time frame for retrieval.

Given the short-term memory characteristics, care must be taken to locate upstream information so that it will not be forgotten by the time the interchange is reached. To ensure this may require repetition if the interchange is a major one or if there is a likelihood for events or features to intervene and extinguish the memory trace.

Reaction Time

Reaction time (RT) is the time it takes a driver to receive a piece of information (e.g., a guide sign), make a decision, (e.g., to take the exit), and take an action (e.g., exit). RT varies from driver to driver and is a function of decision complexity (complex decisions take longer than simple ones) and whether expectancies are violated (*12*). It is easier and faster for drivers to make several simple decisions than one highly complex one, and the long time to process a complex decision takes attention away from other needed information. One way to decrease RT is to use simple, single-decision trailblazers to a destination (e.g., an airplane symbol and an arrow to be followed at each choice point on the route to an airport).

DRIVER ATTRIBUTES AND POPULATIONS

There are more than 160 million licensed drivers in the United States. They encompass a broad spectrum of demographic, physical, and sensory-motor attributes, all of which affect the way they receive and handle information at interchanges. For example, there are differences in sex (more male than female drivers), age (from 14 with no experience to over 80 with decreased capabilities), education (from less than high school to college), and training and experience (from novices to experienced drivers to professionals).

Certain driver subpopulations represent a unique segment of the overall population by virtue of their vehicles (e.g., trucks, motorcycles) or their language fluency (e.g., English, Spanish). These subpopulations can affect safety and operational efficiency, particularly when they represent a large portion of the traffic stream or when they require special treatment or information. Problems that occur with trucks negotiating interchanges and the need for special information is an example of this.

There is variability in driver sensory-motor capabilities, with a considerable range in vision, hearing, and reaction time. Most drivers have static visual acuity corrected to 20/40; approximately 8 percent of the male driving population has color vision deficiencies; and the older driving population (65+) experiences some degree of vision and processing impairments that worsen with age (*4*).

Because people age differently, there is no widely accepted age for the definition of "old." Old is usually considered to be 65 years or older. Currently, 15 percent of Americans are 65 or older. Most older drivers retain their licenses and drive daily, although generally not to work and often not at night.

All drivers ultimately experience age-related sensory-motor impairments that vary from driver to driver. These impairments include a gradual loss of vision and information-handling ability. Common problems include poor night vision and glare recovery, decreased visual acuity, increased reaction time, loss of short-term memory, and poor attention span.

Older motorists compensate by driving slower, avoiding stressful situations, and relying on experience. However, they have a higher-than-average accident rate and are often involved in multivehicle collisions at merges, unprotected left turns, and intersections. Because older drivers use freeways more than they have in the past, more consideration of intersection design and information treatments tailored to older driver attributes is required.

Older drivers can be aided by improved sight distance, enhanced signs and markings, better maintenance of traffic control devices, lower speeds, and alternative means of transport. When the percentage of older drivers in the traffic stream is greater than 15 percent, their diminished capabilities should be taken into account by following these recommendations and by designing for the older driver (*13*).

SIGHT DISTANCE

Drivers must have adequate forward sight distance at interchanges, given their overwhelming reliance on vision for driving and their potentially long reaction times for complex decision making. Adequate sight distance provides sufficient time for drivers to gather information, process it, perform the required control actions, factor in the vehicle's response time, and evaluate the appropriateness of their responses in a feedback process.

Stopping Sight Distance

The Green Book (*14*) defines stopping sight distance as "the forward sight distance available such that a vehicle travelling at or near a highway's design speed has sufficient time to stop before reaching a stationary object in its path." It is the sum of the vehicle's braking time and the 2.5 sec brake-reaction time of an average driver, with a seated eye height of 3.5 ft, for a 6-in. fixed object.

Decision Sight Distance

At interchanges, stopping sight distance may not allow sufficient time for an appropriate, unhurried response, since negotiating an interchange has speed, path, and direction changing components and since stopping is generally not an appropriate maneuver. In addition, drivers often have to make complex or multiple decisions, and there may be visual clutter or violation of expectancies. Longer sight distance is needed to provide more time to detect, recognize, and respond to interchanges. More time is also needed to provide a margin for error if a hazard such as a stopped vehicle or a curb is not immediately detected or recognized, or if information is not present, not properly located, or not readily understood.

Decision sight distance provides longer sight distance and hence more time to see and respond. Decision sight distance is defined as "the distance at which a driver can detect a hazard in an environment of visual noise or clutter, recognize it or its threat, select an appropriate speed and path, and perform the required maneuver safely and efficiently." Decision sight distance can be used to determine the adequacy of forward sight distance to the interchange and to position highway information. The interchange treatments in the *Manual on Uniform Traffic Control Devices* (MUTCD) (*15*) generally provide the necessary sight distance. However, some interchanges may require further analysis. In these instances, the procedure contained in the *Users' Guide to Positive Guidance* (*4*) may be used.

EXPECTANCY

Expectancy relates to a driver's readiness to respond to situations, events, and information in predictable and successful ways. It influences the speed and accuracy of the use of information and is one of the most important human factors considerations in the design and operation of interchanges and information treatments.

Configurations, geometric designs, traffic operations, and traffic control devices that are in accordance with or that reinforce expectancies help drivers respond quickly, efficiently, and without error. Configurations, geometric designs, traffic operations, and traffic control devices that are counter to or violate expectancies lead to longer reaction time, confusion, inappropriate response, and driver error.

Expectancies operate at all levels of the driving task (*16*) with guidance and navigation expectancies most critical to interchange driving task performance. At the guidance level interchange expectancies relate to highway and interchange design, traffic operations in the vicinity of the interchange hazards that may be encountered, and markings and delineation treatments. At the navigation level, interchange expectancies relate to drivers' trip plans (i.e., what route and destination information they will expect at the interchange), their use of route markers and guide signs, their selection of exits at intersections, how they locate destinations and services. They affect route choice and in-trip route diversion, and ultimately, whether motorists arrive at their destinations with a minimum of inefficiency and confusion.

Two types of expectancies are operative at interchanges. The first are long-term, a priori expectancies that drivers bring to the task on the basis of past experience, upbringing, culture, and learning. The second are short-term, ad hoc expectancies that drivers formulate from site-specific practices, interchange designs, signing and marking treatments, and situations encountered in transit.

A Priori Expectancies

Because everyday objects, systems, and information displays are designed to operate in standard, consistent ways and are applied nationwide, certain expectancies are structured over a lifetime. The intent of consistent, standard interchange designs and information treatments as contained in the Green Book (*14*) and the MUTCD (*15*) is to foster rapid, error-free operations. When standard designs are used, expectancies are reinforced, and performance is rapid and error free. When nonstandard designs and information treatments are used, expectancies are violated, and the driver is surprised. The results may include longer reaction time, confusion, inappropriate response, or an accident. In designing interchanges and information treatments, it is necessary to understand the nature of a priori expectancies. For example, because most freeway exits are on the right, drivers expect all exits to be on the right. Unexpected left exits often have serious consequences. Similarly, in rural areas, drivers often expect all interchanges to be simple diamonds. When a cloverleaf or directional interchange is used, driver expectancies are often violated.

Not all a priori expectancies are held by all drivers, given regional and local differences. Thus, if most interchanges in a given area contain left exits, then drivers in that area would expect to exit on the left rather than the right. This expectancy aids performance in the area familiar to the driver, where interchanges are as expected. However, outside that area, the same driver's response would be inappropriate.

Ad Hoc Expectancies

It is important to recognize and understand the nature of short-term ad hoc expectancies structured by in-transit, site-specific situations. As drivers travel through unfamiliar areas, they form ad hoc, site-specific expectancies on the basis of the geometry of the road, interchange designs, and information treatments. For example, if every interchange on a rural freeway is a diamond with a ground-mounted advance guide sign, an ad hoc expectancy will be structured that similar interchanges will be similarly designed and signed. If a downstream interchange is a cloverleaf and no different advance signing treatment is in place to restructure drivers' expectancies, then the ad hoc expectancy of a diamond would be violated and drivers may not respond properly.

In addition, design consistency should be maintained. If upstream road geometry provides a 70 mph design speed with clear sight lines and adequate decision sight distance to the freeway's interchanges, then strangers will expect these design standards to continue downstream. If downstream design standards are lower, or sight distances reduced, driver expectancies will be violated.

Thus, drivers continuously formulate new, ad hoc expectancies on the basis of what they encounter in transit. Engineers and designers should therefore understand and account for each type of expectancy. Both a priori and ad hoc expectancies should be considered in design and operations. Appropriate expectancies should be reinforced and expectancy violations eliminated through the use of consistent, standardized interchange designs and appropriate uniform information treatments. Consistency should be maintained within and between locations and jurisdictions, and it should be recognized that upstream practices affect downstream expectancies.

INFORMATION PRESENTATION PRINCIPLES AND TECHNIQUES

Any information carrier that assists or directs drivers in making speed or path decisions at interchanges aids the guidance task. Information carriers that provide direction and destination information and assist or direct drivers in making directional decisions at interchanges aid the navigation task. All needed information at interchanges should be presented unequivocally, unambiguously, and conspicuously enough to meet decision sight distance criteria, reinforce appropriate expectancies or restructure expectancies that are violated, and enhance the probability of appropriate speed, path, and directional decisions.

Design for Drivers and Target Populations

Information at an interchange should be presented in nontechnical terms because drivers may not understand engineering concepts. It should also be determined whether there are target groups whose needs must be addressed. These groups may be older drivers with vision problems or truck drivers negotiating sharp ramps.

Be Responsive to Task Demands and Driver Attributes

Highway information should convey the operating conditions of interchanges, be responsive to the task demands imposed on the driver by interchange design and geometry (particularly when there are time pressures caused by traffic), and be sensitive to driver sensory-motor attributes. Drivers may become overloaded when they have to process too many sources of information, or when an information source has too much information content. Overloaded drivers may become confused or miss important information sources.

Satisfy All Information Needs

All information needs relative to all aspects of the driving task at the interchange should be satisfied. Speed and path information should always be available. Information needs pertaining to routes, destinations, directions, and services should be displayed when appropriate. Information should be displayed when needed, where required, and in a form best suited to the driver and task.

Maintain Interchange Design and Information System Compatibility

Because drivers formulate driving strategies on the basis of their perception of the design and operations of an interchange, incompatible information displays will lose credibility and may lead to confusion. A determination should be made on how interchange designs and information treatments appear to drivers and whether they are compatible and do not violate expectancies. In the design stage, models or computer simulations could be used to make this determination and to ensure compatibility.

Avoid Surprises

Driver performance is enhanced when forward sight distance provides a clear, unobstructed view of the interchange, its traffic, and its traffic control devices. Problems often occur when drivers are surprised by unexpected or unseen features. If any aspects of the interchange could surprise drivers, advanced warning should be provided.

Eliminate Information-Related Error Sources

Information-related error sources should be eliminated. These sources include missing information; information obscured by foliage, structures, earth berms, dirt, snow, or the like; misplaced information (not in a driver's field of view); devices too close to a choice point; and obsolete, nonstandard, ambiguous, or confusing messages.

Resolve Conflicts When Information Sources Compete

When information sources compete for a driver's limited processing capacity (generally 5 to 9 sources or 2 to 3 bits of information), or when there is a chance of overload, a determination should be made as to what information sources should be displayed, and which should be spread out, moved, or removed. Generally, guidance information relating to speed and path takes precedence over navigation information relating to direction.

Use Spreading

Spreading reduces the chance for overload at high-processing-demand locations by moving less important information sources upstream or downstream, thereby reducing the processing load.

Use Repetition for Interchange Information Treatments

If a time greater than 30 sec to 2 min, a driver's short-term memory span, intervenes between the receipt of advance interchange information and the exit ramp, drivers may forget the message. Repeating the information one or more times will help drivers remember and act on it. Repetition is also useful if an information display may be blocked by foliage or trucks.

Use All Available Navigation Aids and Treatments

Appropriate navigation aids should be used. These aids include overhead signs that can be seen over trucks, oversized route guidance signs to help drivers at choice points, trailblazers to freeways and interchanges, real-time changeable message signs to warn of incidents and help manage congestion, and highway advisory radio, which transmits information into a road user's vehicle (*4*).

CONCLUSIONS

An interchange is the freeway design and operations feature that is most likely to lead to driver error, accident involvement, and driver directional confusion. Consequently, it is important that the design of an information treatment at an interchange is optimized to maximize driver performance and minimize error. One way to achieve this goal is through the application of human factors principles associated with the design and operational features of an interchange.

A number of human factors considerations should be taken into account in freeway interchange design and operations. These include the sensory-motor attributes of drivers, particularly older drivers (age 65+), the way they perform the driving task, the visual capabilities of the driver, the information-handling attributes of the driver, the reasons for driver errors at interchanges, the importance of adequate interchange sight distance, the role of driver expectancy in interchange design, and factors affecting information treatment at an interchange.

The human factors considerations identified here are all important to the safety and operational efficiency of interchanges. It is therefore concluded that engineers and designers should be aware of and account for all of them and bear in mind that their efforts are first and foremost to aid the driver. In this regard, those human factors considerations associated with driver error involvement, expectancy violations, and sight distance criteria are the most critical.

REFERENCES

1. *Highway Statistics 1990*. FHWA-PL-91-003. FHWA, U.S. Department of Transportation, 1991.

2. *Synthesis of Safety Research Related to Traffic Control and Roadway Elements.* FHWA-TS-82-232. FHWA, U.S. Department of Transportation, Dec. 1982.
3. G. J. Alexander and H. Lunenfeld. A Users' Guide to Positive Guidance in Traffic Control. In *Information Design* (R. Easterby and H. Zwaga, eds.), John Wiley, Chichester, United Kingdom, 1984, pp. 351–383.
4. H. Lunenfeld and G. J. Alexander. *A Users' Guide to Positive Guidance* (3rd ed.). FHWA-SA-90-017. FHWA, U.S. Department of Transportation, Sept. 1990.
5. G. F. King and H. Lunenfeld. *NCHRP Report 123: Development of Information Requirements and Transmission Techniques for Highway Users.* HRB, National Research Council, Washington, D.C., 1971.
6. R. M. Michaels. Human Factors in Highway Safety. *Traffic Quarterly*, Vol. 15, No. 4, Oct. 1961, pp. 586–599.
7. J. Reason. *Man in Motion: The Psychology of Travel.* Weidenfeld and Nicholson, London, United Kingdom, 1974.
8. T. H. Rockwell. Driver Sensory Load. In *Proc., National Conference on Highway Operations in the 1980's* (T.C. Helvey, ed.), University of Tennessee at Nashville, Sept. 24–27, 1973.
9. I. Schmidt and P. L. Connolly. *Visual Considerations of Man, the Vehicle, and the Highway.* Publication SP279. Society of Automotive Engineers, New York, March 1966.
10. S. F. Hulbert and A. Burg. Application of Human Factors Research in the Design of Warning Devices for Highway Rail Grade Crossings. In *NCHRP Report 50: Factors Influencing Safety at Highway-Rail Grade Crossings*, HRB, National Research Council, Washington, D.C., 1968.
11. R. W. Cumming. The Analysis of Skills in Driving. *Australian Road Research*, Vol. 9, 1964, pp. 4–14.
12. C. Johannson and K. Rumar. Driver's Brake-Reaction Time. *Human Factors*, Vol. 13, No. 1, 1971, pp. 22–27.
13. *Special Report 218: Transportation in an Aging Society.* TRB, National Research Council, Washington, D.C., 1988.
14. *A Policy on Geometric Design of Highways and Streets.* AASHTO, Washington, D.C., 1984.
15. *Manual on Uniform Traffic Control Devices.* FHWA, U.S. Department of Transportation, 1988.
16. G. J. Alexander and H. Lunenfeld. *Driver Expectancy in Highway Design and Traffic Operations.* FHWA-TO-86-1. FHWA, U.S. Department of Transportation, May 1986.

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Interchange Study and Selection Process

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The ability of the public to use a highway facility to the capacity level for which the roadway was designed will depend to a great extent on the accommodations made for intersecting traffic patterns. The use of grade separations and interchanges can result in improved operations on both roadways. However, because of the investment required to provide grade separations, the selection of the highest priority locations and the design of the interchange are critical. One method of determining the priority of competing interchange locations is through a cost-benefit analysis. The Texas Transportation Institute developed a computer analysis program called Texas Ranking of Interchange Projects (TRIP) to accomplish a relative analysis of interchange cost reductions on a macroscopic level. The major economic benefits of an interchange include a reduction in delay costs, a reduction in vehicle operating costs, and the expected accident cost reductions. These benefits are summarized and matched against the expected construction and right-of-way costs of the proposed interchange. The selection of the interchange type is influenced by many external constraints. These constraints may include limited right-of-way, environmental considerations, historical structures, and handling traffic under construction. Interchange types include trumpets, diamonds, cloverleafs, and directionals. Each interchange type has specific advantages and disadvantages. The design selected must be constructible and maintainable under traffic and must be able to handle traffic movements into the ultimate design year. Experienced design, comprehensive research, and knowledgeable drivers are necessary to achieve the maximum highway user benefits from these facilities.

The ability of the public to use a highway facility to the capacity level for which the roadway was designed will depend to a great extent on the accommodations made for intersecting traffic patterns. When these intersecting traffic patterns can be grade separated, the resulting roadway will function with increased efficiency, improved operations, and a higher degree of safety. However, it is not practical or cost-effective to grade separate all intersecting traffic movements. Since interchanges represent the most costly intersection treatment in terms of initial investment, the selection of the highest priority intersection locations and the design of the interchange are critical in providing an effective and efficient highway system. An example of the order of the interchange selection and design process is shown in Figure 1.

An interchange is defined by AASHTO as a system of interconnecting roadways in conjunction with one or more grade separations, providing for the movement of traffic between two or more roadways on different levels (*1*). Interchanges may be included in new location projects or in facility upgrade projects. Reconstruction work may improve an existing grade separation or provide a new interchange at the at-grade intersection of two or more roadways.

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Requirements such as structures, embankments, and rights-of-way contribute significantly to the cost of an interchange. Whereas simple interchanges may cost a few hundred thousand dollars, it is not unrealistic for a fully directional freeway-to-freeway interchange in an urban area to cost more than $100 million. The decision to build an interchange and the type of facility to construct will have a significant impact on the financial resources of a transportation agency.

Each transportation agency is faced with limited funds to design, construct, and maintain the highway system within its jurisdiction. The agency must first make a commitment to maintain the infrastructure currently in place. The remaining resources can be used to improve, expand, or relieve congestion within the existing network. An interchange accomplishes these goals through improved highway safety, expanded traffic operations in the area of the intersection, and relief from existing roadway congestion.

Because resources are limited, the locations with the greatest potential for improvement must be given priority. In order to maximize highway user benefits, ranking methodologies are used to identify proposed interchange locations with the greatest potential for increasing user benefits and reducing user costs.

After the intersections needing improvement are ranked, the application of a particular interchange configuration begins to come into focus. Present and future traffic movements must be identified. The functional classifications of the intersecting roadways will often give preference to one of the roadways in the design. Socioeconomic constraints should be identified as early in the planning and design phase as possible to avoid future conflicts or design changes.

Methods for determining interchange need and setting priorities will be discussed in more detail in the following sections. Some of the considerations in determining interchange configurations will also be presented.

SYSTEM VERSUS SERVICE NEED

Even though interchanges are, of necessity, designed to fi individual conditions and controls, it is preferable that th pattern of interchanges, ramps, and access control along freeway follow some degree of consistency (*2*). Factors suc as the number of lanes, signing, lane balance, and route co tinuity also contribute to this consistency. To accomplish th consistency, an interchange design must take into account total highway system approach. Although this has always bee true in initial roadway construction, it is becoming increa ingly important for reconstruction projects. The sheer ma nitude of a major interchange project will affect traffic mov

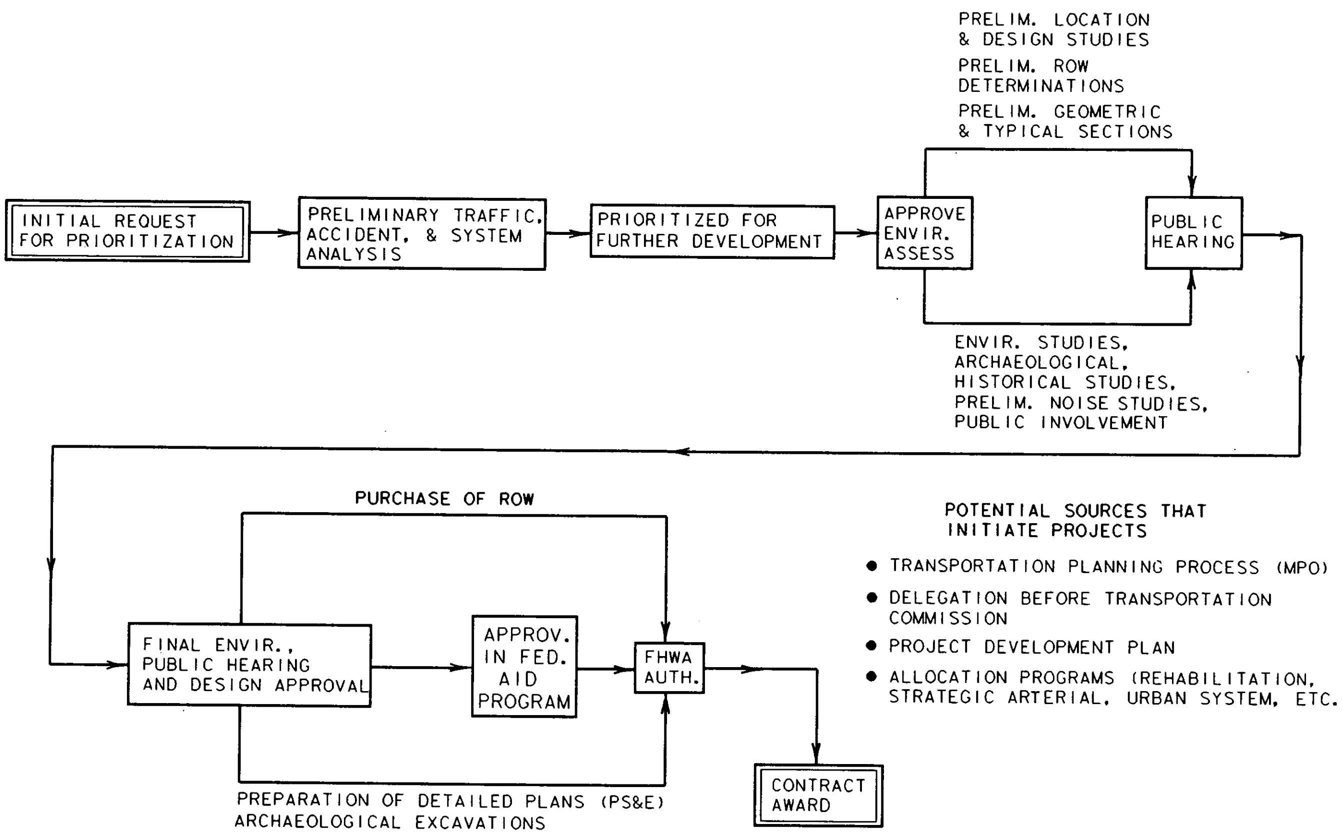

FIGURE 1 Interchange selection and development process.

nents far upstream and downstream on the through roadways nd will have a significant impact on adjacent land use.

Interchanges may contribute to a system operation or a ervice operation. Generally, system interchanges provide for reeway-to-freeway connections, whereas service interhanges connect one level of the highway system to a lowerervice-level facility. The design of service interchanges may e influenced by the functional classification of the roadways, ith priority given to the higher classification.

A system approach to interchange need will often dictate itial interchange placement. For example, freeways usually equire grade separations at existing state system routes. If ufficient traffic demands are evident, county road crossings ay also indicate the need for a grade separation. The extent nd complexity of these separations will depend on present nd future traffic demands.

A service approach to interchange need indicates a regular attern of access to the through roadways. Where existing outes do not dictate the need for interchanges in rural areas, e need for a freeway grade separation may be related to ser delay. Urban conditions may relate interchange need to affic demand or spacing frequency.

A commonly used guide for freeway interchange spacing is mi in urban areas and 3 mi in rural areas. These spacings e often changed to meet demand and, in urban areas, may changed through the provisions of collector-distributor roads d grade-separated ramps. Urban area interchanges may so require detailed traffic projections, including turning movements and capacity analysis, before the alternatives can be narrowed to a final design.

INTERCHANGE NEED PRIORITIES

When a transportation department faces a limited pool of resources, proposed interchange locations must be analyzed to determine which sites merit the highest priorities. These needs should be outlined as early in the program planning stage as possible. These decisions suggest the use of a consistent ranking process. One method of ranking these projects is through the use of a benefit-cost analysis. Unfortunately, there is often little detailed operational data available in the early programming phase. A benefit-cost analysis is necessary to establish preliminary project priorities. This preliminary analysis can evaluate simple road user costs to allow an early planning stage analysis of design alternatives.

Several computer programs are available to analyze the more complex operational aspects of intersections and interchanges. Some examples of these programs include PASSER-II, TRANSYT-7F, and NETSIM. However, these programs generally require a significant amount of data on traffic operations to produce accurate output. At the programming stage, this level of traffic detail is rarely available. A macroscopic modeling procedure is usually necessary to compare competing interchange location alternatives.

The Texas Transportation Institute developed a computer analysis program called Texas Ranking of Interchange Projects (TRIP) to accomplish a relative analysis of interchange cost reductions on a macroscopic level (*3*). Although TRIP cannot produce a finalized ranking of priorities, the program goes beyond simple road user costs and summarizes the major cost savings associated with project development to make comparisons between similar interchange requests. The major economic benefits, or cost reductions, associated with interchange development can be grouped in three categories.

The first benefit to be analyzed is the reduction in delay cost associated with providing a grade-separated interchange or upgrading the interchange design. Figure 2 shows a comparative summation of intersection delay for various interchange configurations (*3*). Although delay can be used strictly for a relative comparison of the improvement, this savings can also be used as an actual numerical measure by placing a value on this delay time. At the programming stage, only the most significant traffic characteristics can be compared in delay reductions. The analysis makes a comparison of the movements on the at-grade intersection against the movements resulting with the grade separation and the remaining operations occurring at-grade. The difference in delay represents the savings realized through delay reductions. An example of system delay curves for a simple diamond interchange is shown in Figure 3 (*3*).

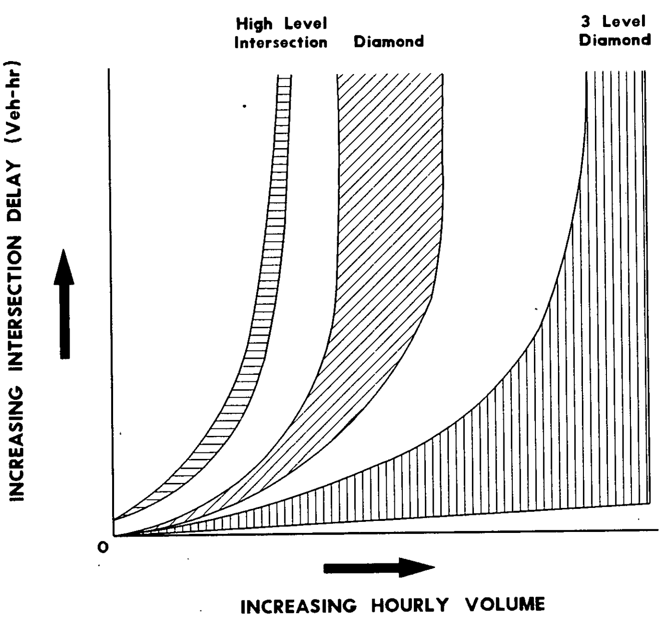

FIGURE 2 Example of delay versus capacity (*3*).

The second cost area to be analyzed is the reduction in vehicle operating cost. Estimates are made of running costs for vehicular travel, slowing and stopping at intersections, and idling while waiting to execute a particular movement. These operating costs are summarized for both before and after conditions to determine the savings in vehicle operating costs due to grade-separated movements at an interchange.

Finally, an analysis is performed to determine the expected accident cost reductions. Accident rates are estimated for both the existing intersection and proposed interchange. Coded accident data often make it difficult to distinguish between different existing intersection configurations in comparative assessments. The total accident costs will be the cost per accident multiplied by the accident rate. Significant differences exist in cost per accident figures, but application of a consistent set of figures will make a relative comparison valid.

These three major cost reductions are summed to give an early planning stage estimate of the gross benefits to be realized from interchange development. These benefits will be matched against the total construction cost of the interchange including any needed right of way. Although the complete design will not have been determined at this stage, any proposed interchange layout must adequately satisfy the present and future traffic demands. These locations with the highest benefit-cost ratios will be placed at the top of priority lists for further consideration and development.

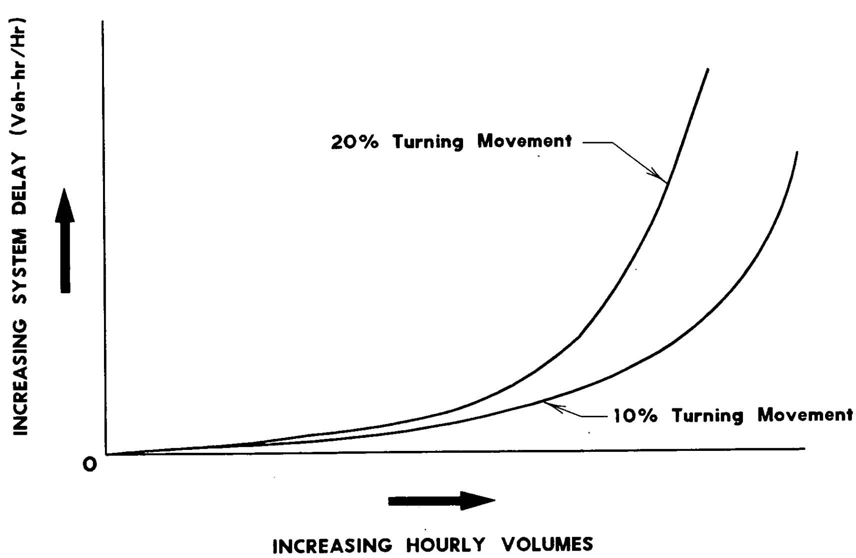

FIGURE 3 Example of system delay curves for a simple diamond interchange (*3*).

CONSIDERATIONS IN INTERCHANGE SELECTION

Traffic-handling techniques have improved considerably since the days of the intersection shown in Figure 4. However, many of the design constraints of yesterday still exist and are magnified in today's urban and rural environments. Interchanges designed to meet the best possible geometric and operational configurations with no outside limitations would represent an ideal design situation. However, interchanges are rarely designed strictly for geometric and operational considerations without external constraints. These external constraints may be numerous and vary considerably from project to project.

Right-of-way restrictions are some of the most common conditions that influence the ultimate interchange layout. Some of these restrictions in right-of-way may be manmade, such as developments or office buildings. If the development is significant enough, then it may not be economically feasible to obtain the right-of-way. In developed urban areas, right-of-way costs have exceeded $100/ft² in some cases. This right-of-way cost still does not include applicable damages to remaining property or damages for lost or reduced access during construction. The natural terrain of the right-of-way may limit development of a complete interchange in one or more quadrants. Existing roadway conditions may play a major part in determining the operational and design options available for the proposed interchange locations. The presence or absence of frontage roads within the right-of-way may also have a major influence on the final layout.

Environmental considerations are becoming an increasing concern to the public in all highway construction. Interchanges are environmentally scrutinized to even greater lengths because of the large land area that they occupy and the concentration of traffic in the area. Noise levels around the interchange are estimated, and mitigation factors may be necessary if neighborhoods will be disturbed. Storm water run-off from the interchange area may have to be filtered if contaminant levels exceed recommended limits. Wetlands areas can generally not be taken unless they are replaced in kind. Historical abutting property structures should be identified early in the planning phase to avoid conflicts with the proposed interchange. The public is demanding that transportation agencies be environmentally accountable and reduce the potential impacts of highway construction and traffic operations to the greatest extent possible.

The geometrics of intersecting roadways also specify some of the conditions of the design selected. The selection of the prioritized traffic movements and the roadways to be carried

IGURE 4 Circular intersection.

over on structures will dictate subsequent design decisions. Considerations such as topography, drainage, and economics will influence the choice between overpasses and underpasses.

The selection of particular interchange designs, as shown in Figure 5, will also depend on the merge and diverge opportunities that are needed to handle traffic demand. If a particular turning movement has significant traffic volumes, then the chosen design must be capable of handling the needed capacity. Also, if an individual turning movement has a high percentage of trucks, then the geometrics of the connectors or ramps must be liberally designed with respect to curvature and superelevation to allow efficient operation. In particular, the combination of a downgrade with a sharp turning maneuver against adverse superelevation is believed to increase the overturning tendency of large trucks (*4*).

Of particular concern in urban areas, where interchanges are being rehabilitated or reconstructed, is the ability to accomplish the proposed construction under traffic. Coordinated traffic handling is necessary to ensure the safety of the traveling public as well as the safety of construction personnel. Signing and pavement markings, both during construction and after completion, are critical to the expected performance and capacity of an interchange design. The old saying, "If you can't sign it, don't build it," is still a good rule. With increasing traffic volumes, more elderly drivers, and greater vehicular size differentials, safe and efficient traffic operations depend on a uniform and identifiable system of signing and markings.

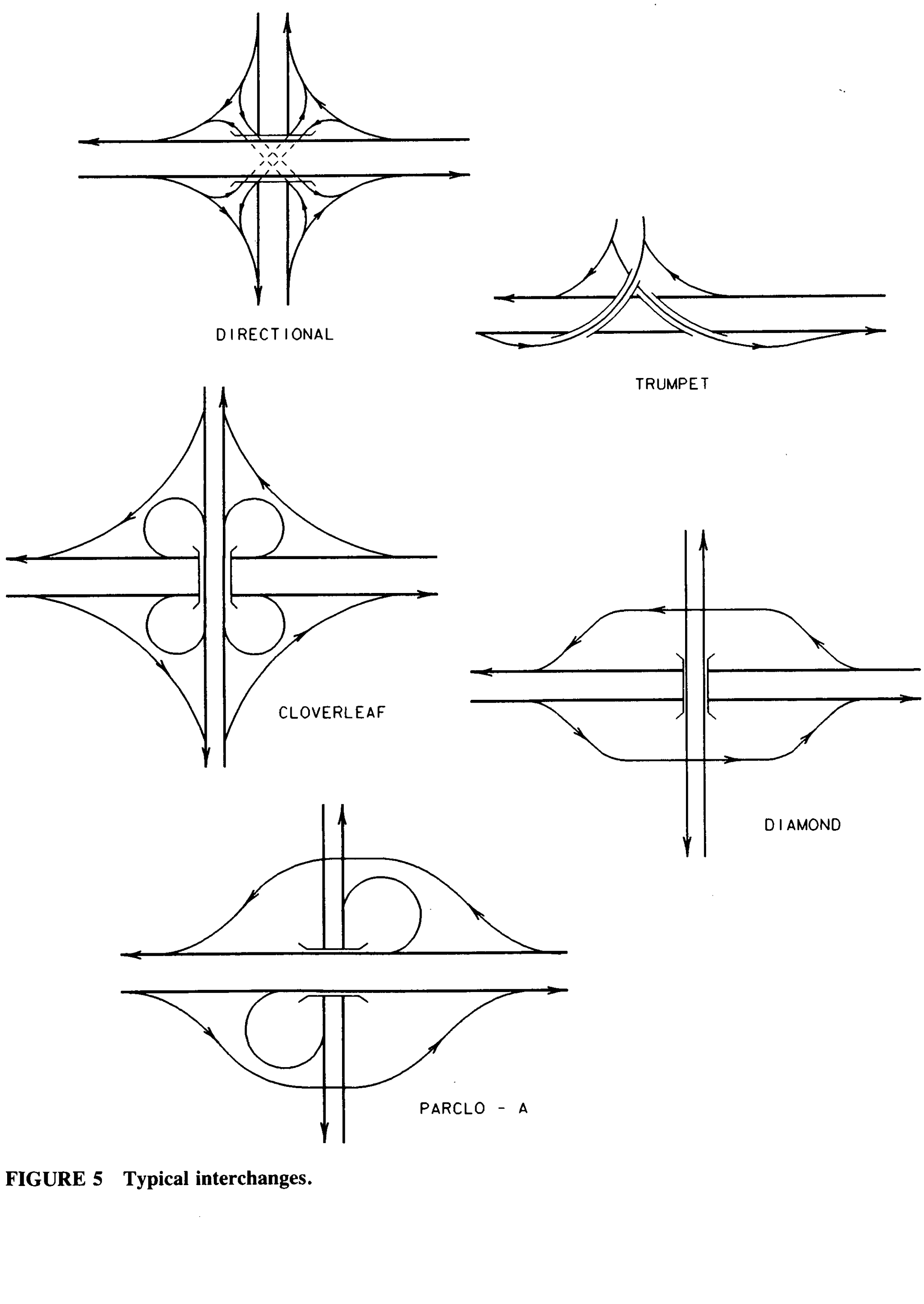

FIGURE 5 Typical interchanges.

WEIGHING ALTERNATIVE DESIGNS

The actual interchange layout will depend on the traffic movements that must be economically processed through the interchange. Carefully selected traffic objectives need to be outlined before selection or elimination. Each interchange configuration has advantages and disadvantages, which cover a wide range of conflicting areas of need. Comparisons and decisions often have to be made among issues that are not directly related.

When one roadway terminates at another through roadway, a trumpet interchange with one or more grade separations may be selected. These interchanges are referred to as T-type if the third leg intersects a through roadway at right angles. If the intersecting roadway forms an acute angle, the interchange is referred to as a Y-type. Although the difference between these types is not significant, the three-leg intersections show wide variations in interchange designs.

Loop ramps for trumpet interchanges are used to make connections between the through roadway and the terminal leg. The design should give preference to the turning movement with the greatest volume. The use of an oblong shape with appropriate superelevation for the loop ramp allows for better operations under high-volume and high-speed conditions. Depending on the level of service desired, the interchange may necessitate increased amounts of bridge structure. Also, where traffic weaving is necessary, provisions for sufficient ramp length at the merge and diverge points should be analyzed early in the design phase. Generous loop radii and merge lengths will contribute to efficient and safe traffic operations.

Four-leg interchange designs have multiple variations. One of the most common types is the conventional diamond interchange. The conventional diamond is simple and economical to construct, can provide relatively high speed entrance and exit ramps, and requires a relatively narrow strip of right-of-way (*5*). This configuration is applicable to urban and rural conditions, particularly where a major highway crosses a lower classification facility.

Conventional diamond interchanges also have several shortcomings. These configurations have double four-leg intersections; two of the approaches represent one-way operations. This may result in a potential traffic operations problem if wrong-way entry occurs at one of the intersections. The diamond is also limited by the amount of traffic that can be cycled through the two at-grade intersections. The use of a turnaround lane, to allow drivers to make u-turns without going through the at-grade intersections, can enhance the capacity of the diamond interchange.

Different versions of the diamond interchange, as shown in Figure 6, can enhance the design for selected situations. The spread diamond is one example. The spread diamond moves the ramp connections to the crossroads out far enough to allow future loop ramps inside the connections. These loop ramps will then accommodate left-turning vehicles. Unfortunately, this configuration requires considerably more right-of-way. In addition, after loop operations are added to a spread diamond configuration, the interchange features a double exit design and generally short radius loop ramps.

The split diamond interchange allows for the off-ramp connection at one crossroad while the on-ramp connection occurs beyond a succeeding crossroad. This adaptation limits access and contributes to better mainline operations. This design is particularly applicable in urban areas with narrowly spaced crossing streets. However, business owners and the traveling public may object to the chosen access points. Public involvement and careful traffic engineering analysis are necessary to provide appropriate split ramp connections.

The three-level diamond interchange allows both through roadways to cross on their own alignments with turning movements separated on their own level. Careful planning in the initial design phase can allow this configuration to be converted to a directional interchange during a subsequent construction project. Three-level diamonds are historically not recommended for ultimate design due to the limitation of turning capacities that can be handled on a single level. However, properly designed three-level diamonds operated satisfactorily for many years before the ultimate directional interchange was necessary.

Another common type of interchange configuration is the cloverleaf interchange shown in Figure 7. The full or partial cloverleaf interchange accommodates left-turning traffic through the provision of loop ramps. Although cloverleafs are more expensive than diamond interchanges, they have been used effectively in rural or lightly developed areas. Collector-distributor roads enhance the ability of a cloverleaf to handle a significant traffic volume by eliminating the double exit and removing the weaving maneuvers from the main roadway.

Cloverleaf interchanges have several disadvantages from a traffic operations standpoint. These disadvantages can be mitigated to a great extent by generous design and signing if established early in the planning stage. The use of loop ramps to accomplish a left-turn movement requires some misdirection on the part of the driver. The length of the loop and the required right-of-way will increase quickly as the design speed is increased. Larger trucks do not operate well in loop configurations, particularly if they have large off-tracking characteristics. The loop ramps in adjoining quadrants have a capacity limit of about 800 to 1,200 vehicles per hour, depending on the loop configuration and the level of service considered acceptable. Once the capacity limit of the loop ramps is reached, these inefficient ramp operations begin to affect traffic on the through roadway. Also, loops are generally limited to single lane operations (*6*).

Directional or semidirectional interchanges, as shown in Figure 8, represent the highest type of interchange and are generally reserved for freeway-to-freeway movements in urban conditions. Some interchanges will only require a direct connection for one particular left-turn movement. Directional ramps improve traffic operations by reducing travel distance and increasing speed and capacity. Directional ramps also eliminate many of the weaving maneuvers necessary in other configurations. If these connections are liberally designed, the ramps can approach the capacity of an equivalent lane on the through roadway.

Fully directional interchanges are, however, the most expensive type of single highway feature constructed. These designs also require significant amounts of high-cost right-of-way, which the public is increasingly cautious about converting to highway usage. These facilities are usually phase constructed under high traffic conditions, which significantly lengthen construction time and irritate owners of adjacent

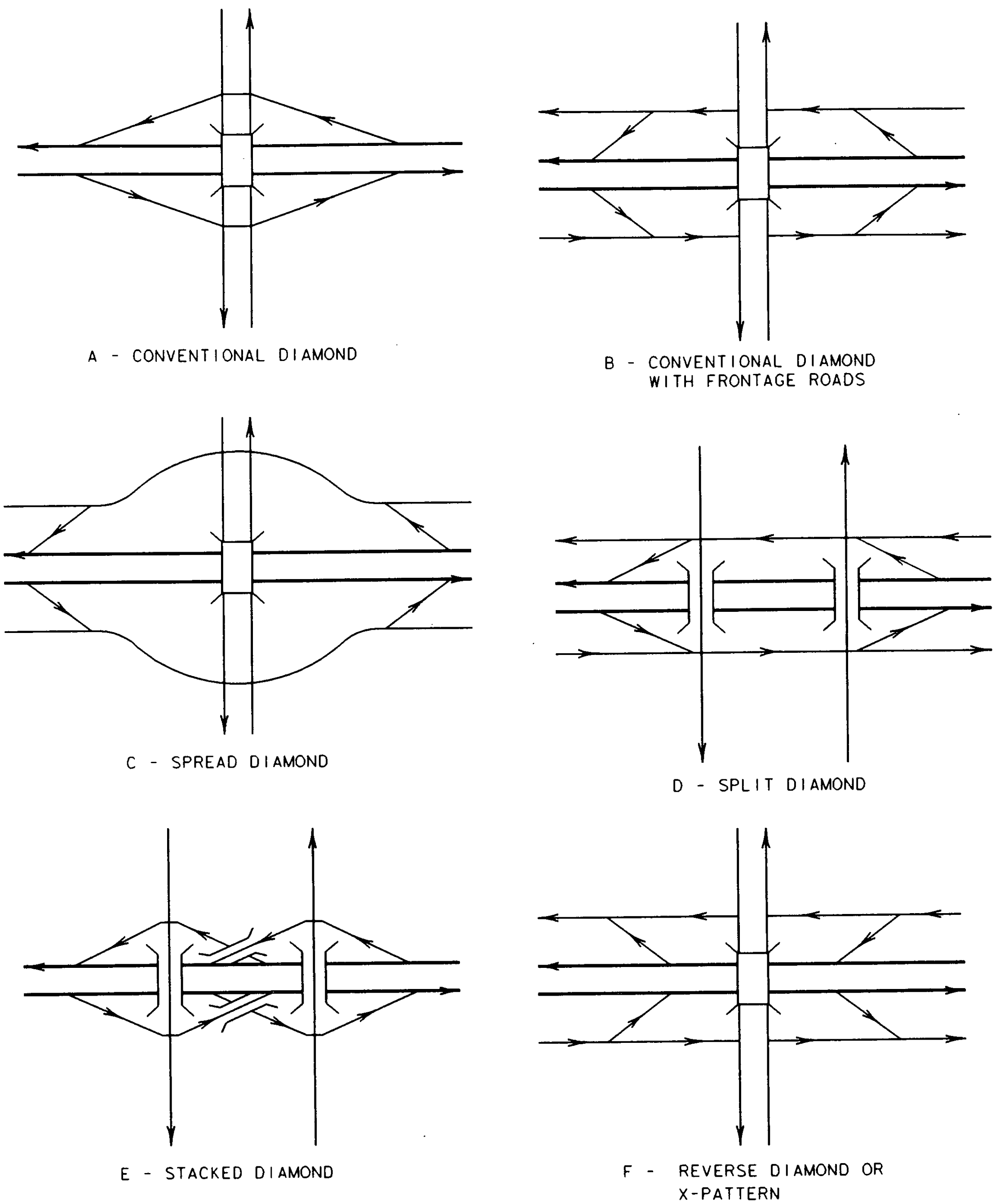

FIGURE 6 Variations of diamond interchange.

businesses and the traveling public. These interchanges are so expensive that a state district office, covering many counties and a major urban area, may find that the cost of one fully directional interchange exceeds the entire yearly construction allocation. Detailed traffic analysis and multiple design alternatives necessitate significant predesign study and public input before commitments are made to a fully directional interchange project.

Ramps and connections for all interchange designs represent a continuing source of operational difficulties and potential accident locations. This is due in part to the multiple actions and decisions required simultaneously by the driver. The driver's multiple task scenario can be further complicated if the geometrics of the ramp are restrictive, signing is not informative, advance notice of the decision point is not given, or the ramp is hidden by physical features.

Methods of overcoming these difficulties include designing ramps that satisfy driver expectancy. Single exits contribute to this expectancy and make signing much simpler to accomplish. An example of the use of single entrance and exit design is shown in Figure 9. Ramp operations may also be improved by moving the relatively sharp ramp curvature away from the ramp terminal (7). When multilane ramps or connections are necessitated by traffic volumes, the touchdown point on the

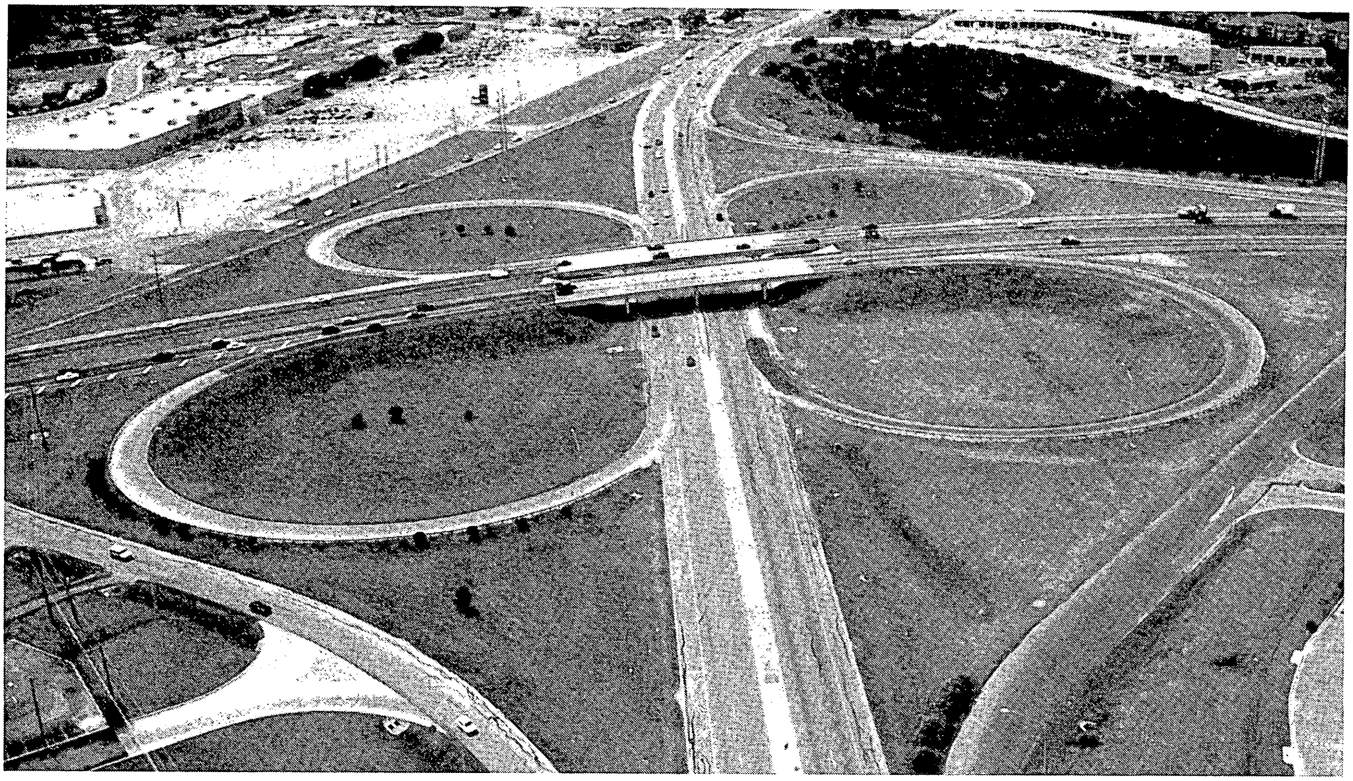

FIGURE 7 Cloverleaf interchange.

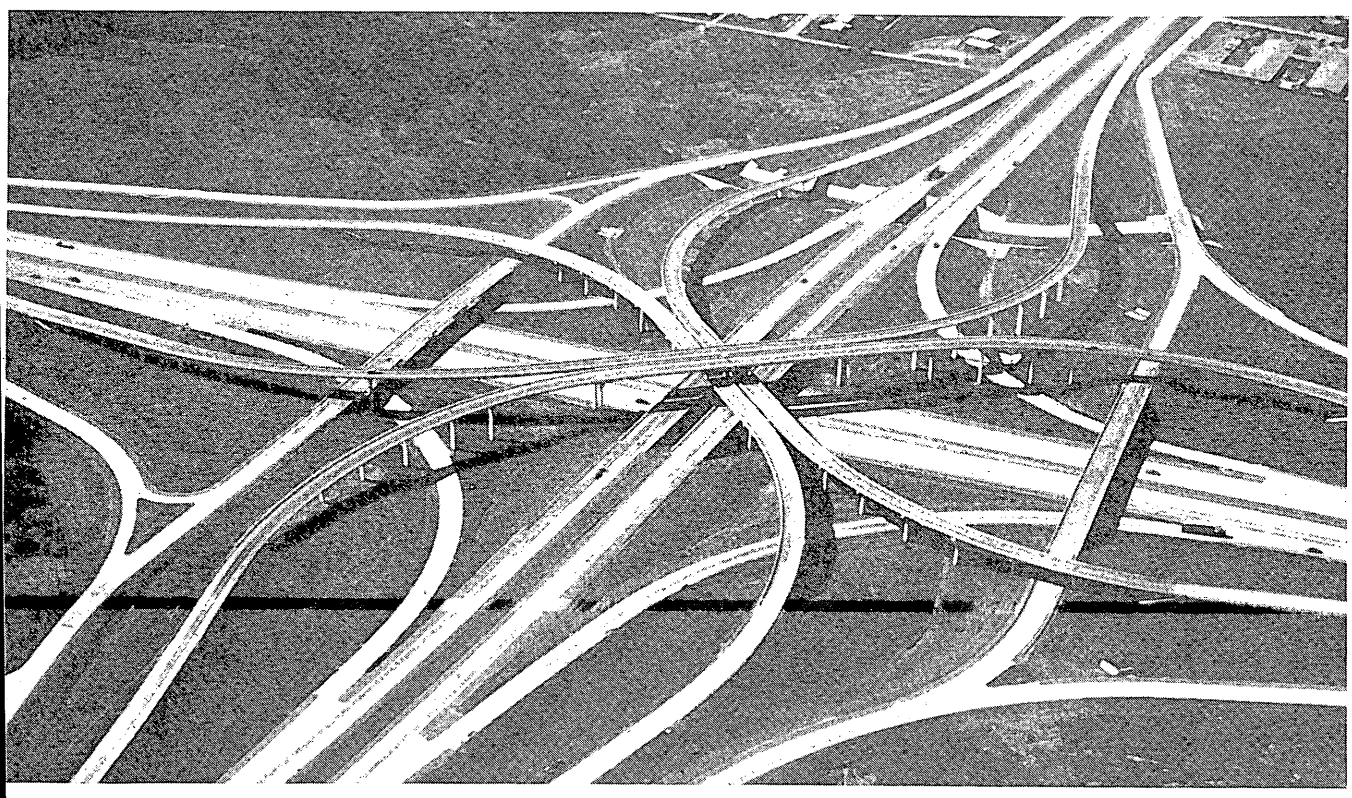

FIGURE 8 Directional interchange.

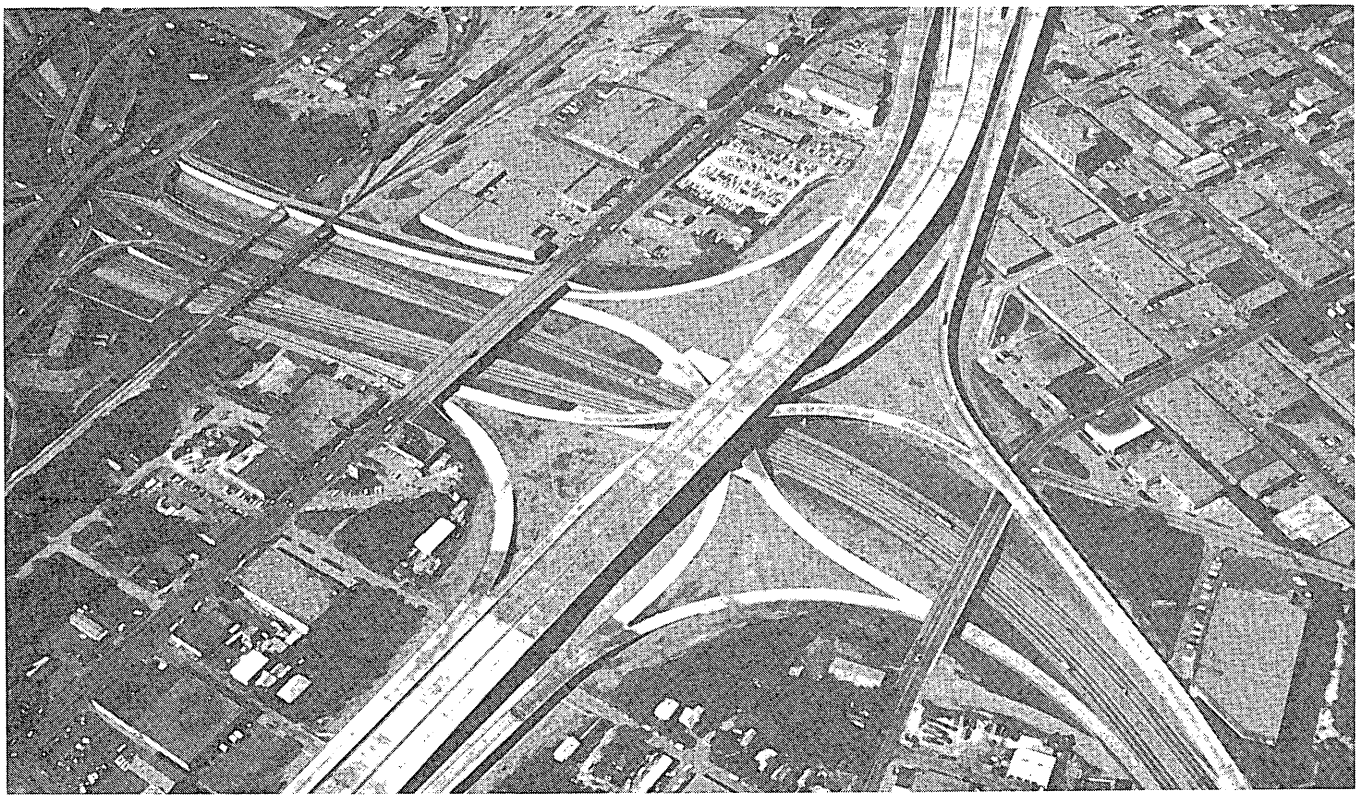

FIGURE 9 Directional interchange with single entrance and exit design.

through roadway should be able to accommodate all lanes of the roadway and the ramp for a considerable distance downstream of the terminal. If ramp lanes are forced to merge at or before the terminal, then operations will deteriorate before the design capacity of the ramp is reached, and the benefit of the multilane connector will be lost. The use of auxiliary lanes and following lane balance design can eliminate many of these deficiencies.

CONCLUSION

Because of the complex nature of the design process and the magnitude of the cost of construction, interchange development in the future must be done on the basis of specific needs. The limited resources available to transportation agencies will require a documented prioritization process for the expenditures necessary to make capacity improvements at major roadway crossings. Because highway and interchange design is rarely done without external constraints, several contributors to the process must take a more active part in the design and final implementation of a safe and cost-effective highway facility. Experienced designers, comprehensive researchers, and educated drivers all contribute to the ultimate success of the transportation system.

Design efforts need to take full advantage of the experience gained in the development of the original Interstate system (*8*). Interchange development and expansion of the future demands an economical and efficient design approach. The reconstruction process will doubtlessly be more complex than the original system construction. Designers must build on the foundation of that experience and use the best possible design techniques of the future to meet the transportation needs of a complex and changing society.

Research efforts must take on a broadened vision of interchange design. Future research efforts need to probe deeper than the simple "longer or wider is better" approach. The interaction of the individual components of an interchange design must be studied in unison. Interchange improvements must also be examined within the concept of the entire configuration, the total roadway system, and a realistic set of typical external constraints under which these facilities have to be constructed. The public will continue to be concerned about the environmental impacts and the spending of tax revenue associated with maintaining and improving the national transportation system. Suggested ideas and improvements need to give proper consideration to all these factors to be of practical value.

Efforts to educate the users of increasingly complex highway systems and interchange configurations must be continued. Some of these efforts should be passive from the driver's perspective. They include uniform signing and consistent entrance and exit terminal design at all interchanges regardless of configuration. Some efforts need to be more direct, such as the Give Them a Brake campaigns, to promote driver safety in construction zones.

Improvement in some of these areas, in conjunction with the best possible selection and design of interchanges, will ensure maximum highway user benefits from these facilities.

REFERENCES

1. *AASHO Highway Definitions.* AASHTO, Washington, D.C., June 1968.
2. *Highway Design Manual* (4th ed.). California Department of Transportation, Sacramento, July 1990.
3. J. L. Memmott et al. *Texas Ranking of Interchange Projects—TRIP, PC Interchange and RR Grade Separation Benefit-Cost Program.* Research Study 2-8-87-1105. Texas Transportation Institute, Texas A&M University, College Station, Nov. 1988.
4. *Manual of Policies and Procedures.* Illinois Department of Transportation, Springfield, May 1988.
5. *A Policy on the Geometric Design of Highways and Streets.* AASHTO, Washington, D.C., 1990.
6. *Operations and Procedures Manual.* Texas State Department of Highways and Public Transportation, Austin, Feb. 1987.
7. J. E. Leisch. Planning/Design Features and Case Studies in Freeway Reconstruction, Part I. *ITE Journal*, Washington, D.C., Oct. 1983.
8. J. E. Leisch. Planning/Design Features and Case Studies in Freeway Reconstruction, Part II. *ITE Journal*, Washington, D.C., Nov. 1983.

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Accidents and Safety Associated with Interchanges

James M. Twomey, Max L. Heckman, John C. Hayward, and Richard J. Zuk

The primary objective of this study was to critically review, summarize, and document past safety research that associates accidents and safety with interchange features. Geometric layout, including alignment, ramp types, and interchange areas, and the effects of spacing between interchanges as they relate to accidents are discussed. Collectively, research results indicate that interchange ramps should be designed with flat horizontal curves (except in rural areas), and the maximum degree of curvature for a given design, speed, and superelevation should be avoided. Sharp curves at the ends of ramps and sudden changes from straight alignment to sharp curves should be avoided as well. Ramps of all types and sizes can be designed to connect two or more legs at an interchange. In summary, studies indicate that the design of cloverleaf ramps, scissor ramps, and left-side ramps should be avoided where possible. Collector-distributor roads should be considered in high-volume interchange designs and especially designs in which loop and cloverleaf ramps are used. Interchange areas include the areas along the freeway mainline between and including acceleration lanes, deceleration lanes, and their respective ramps. The relative safety of entrance and exit terminals is enhanced with geometric designs that provide 800-ft or longer acceleration or auxiliary lanes. The same is true for weaving lengths. The potential for accidents has been related to the volume of the ramp and through-lane traffic volumes. Interchange accident rates have been shown to increase as interchange spacing decreased in urban areas. It is concluded that interchange rehabilitation projects are effective in reducing accident experience.

An interchange is a system of interconnecting roadways that provides for movements between two or more grade-separated highways. This paper is focused on safety research related to interchange design. Interchange safety relates to how the interchange operates within the overall highway system and how the components of an interchange are interrelated.

In the overall highway system, the key elements of interchange safety research relate to interchange configurations, traffic controls, and spacing. Many interchange configurations are defined in the AASHTO Policy on Geometric Design of Highways and Streets, including cloverleafs, diamonds, trumpets, and directionals. Variations of each of these types are also defined, resulting in a total of 12 or more interchange types.

Safety research has been focused primarily on the most common types: diamonds and cloverleafs. Geometric safety research on individual interchanges has been focused on ramps, ramp terminals, speed change lanes, alignment, and spacing.

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Ramp safety elements include acceleration lanes, deceleration lanes, weave sections, ramp alignment, and ramp terminals. Interchange alignment factors include grades, curves, vertical and horizontal clearances, and sight distance.

Geometric layout, including alignment, ramp types, and interchange areas, and the effects of spacing between interchanges as they relate to accidents are discussed here. Accident data and research results are presented to aid planners, designers, and decision makers in the implementation of safe highway design. This information can be used in the design of new interchanges and the increasingly important redesign of older interchanges that do not meet current needs.

SUMMARY OF RESEARCH

Alignment

Interchange alignment, specifically ramp geometry, at a particular site is determined by many factors. These include the number of intersecting legs, traffic volumes, topographic and environmental setting design controls, and their consistency with the overall roadway system they serve. Both horizontal and vertical alignment have been considered in safety research.

Horizontal Alignment

Horizontal alignment of ramps has been the subject of several safety studies in the past. The primary results of the studies have shown that (a) except for loop ramps in rural areas, all right-side and outer-connection ramps showed an increase in accident rates with increasing maximum curvature, and (b) outer-connection ramps in urban areas tend to show increasing accident rates with increasing average daily traffic (ADT). Straight outer-connections have lower accident rates than curved connections in urban and rural areas for all ADT levels, except 0 to 499 in urban areas (Table 1) (*1*).

Rural loops with low curvature have higher accident rate than rural loops with high curvature, whereas the reverse is true for urban loops (Table 2) (*1*).

Accident rates are grouped by ramp types and curvature in Table 3 (*2*). Off-ramps have the highest accident rate, which can be attributed to high speeds of vehicles entering ramp curves and ramp terminal capacity deficiencies.

TABLE 1 Accident Rates on Outer Connections by Curvature and ADT (*1*)

ADT	Urban[a]		Rural[a]	
	Straight <1°[b]	Curved >1°[c]	Straight <1°[b]	Curved >1°[c]
0-499	0.74	0.64	0.00	0.67
500-1000	0.34	0.72	0.13	0.49
1001-1500	0.64	0.84	0.00	0.61
1501-2000	0.15	0.93	0.00[d]	0.20
>2001	0.49	0.82	0.00[d]	0.72
all volumes	0.44	0.81	0.05	0.56

[a] Accidents per 100 million vehicles.
[b] Less than 1 degree of curvature.
[c] Greater than 1 degree of curvature.
[d] Less than 10 units.

TABLE 2 Accident Rates on Loops by Curvature and ADT (*1*)

ADT	Urban		Rural	
	Low[a] <12°[b]	High[a] >36°[c]	Low[a] <12°[b]	High[a] >36°[c]
0-499	0.000[d]	0.841	1.000	0.26
500-1000	0.000[d]	0.960	0.810	0.37
1001-1500	1.320[d]	0.690	0.000[d]	0.00
1501-2000	0.000[d]	0.720	0.000[d]	0.00
>2001	0.141	1.000	0.000[d]	0.00
all volumes	0.200	0.940	0.631	0.25

[a] Accidents per 100 million vehicles.
[b] Less than 12 degrees of curvature.
[c] Greater than 36 degrees of curvature.
[d] Less than 10 units.

Vertical Alignment

Ramp grades are generally constrained by the location of the crossing (intersecting) route, either overcrossing or undercrossing. The results of one study, classified by undercrossing and overcrossing accident rates by ramp type, are presented in Tables 4 and 5 (*2*). Trumpet ramps, cloverleaf ramps, loops without collector-distributor roads, and left-side ramps have consistently higher accident rates than their counterparts, regardless of upgrade or downgrade. Overall, however, on-ramps have been found to have the same combined accident rates for downgrades and upgrades. Uphill off-ramps, however, have lower combined accident rates than downhill off-ramps.

Collectively, research concludes that interchange ramps should be designed with flat horizontal curves (except in rural areas), and the maximum degree of curvature for a given design, speed, and superelevation should be avoided. Sharp curves at the ends of ramps and sudden changes from straight alignment to sharp curves should be avoided as well. The crossing routes should be over the intersecting freeway based on safety, lower construction costs, and easier future mainline freeway traffic control during reconstruction.

Ramp Type

Ramps of all types and sizes can be designed to connect two or more legs at an interchange. Ramps provide the connection between crossing routes. Correlations have been developed between accident rates and types of freeway ramps (Table 6) (*2*). Left-side ramps and scissor ramps have much higher accident rates than other types, and their use is now generally discouraged. Diamond ramps have the lowest rate, but these rates do not account for crossroad and ramp intersection accidents.

Recent studies of the geometric design of ramps on which a high rate of truck accidents occurred concluded the following: (a) truck loss-of-control accidents on ramps are predominantly rollover and jackknife accidents; (b) jackknife accidents occur predominately at sites where inadequate pavement friction levels prevail during wet weather; (c) truck rollover accidents occur on ramps on which the trucks are traveling faster than the design speed of the ramp; (d) in designing horizontal curves to accommodate trucks, it is important to check for both rollover and skidding potential to determine which controls the design, and (e) the AASHTO policy of accepting ramp downgrades as high as 8 percent may be ill advised at sites at which an actively sharp curve remains to be negotiated toward the bottom of the grade (*3*).

TABLE 3 Accident Rates by Ramp Type and Curvature

Ramp	No. Ramps	No.[a] Accidents	MV[b]	Accident Rate[c]
On-ramps Straight	180	282	524.5	0.54
Curved	150	229	335.2	0.68
Off-ramps Straight	188	420	536.0	0.78
Curved	142	258	310.1	0.81
Total on & off Straight	368	702	1060.5	0.66
Curved	292	487	645.3	0.75

[a] No. of Accidents
[b] Million Vehicles.
[c] Accidents per Million Vehicles.

TABLE 4 Ramp Accident Rates by Ramp Type, Overcrossing (2)

Type of Ramp	ON				OFF			
	No. Ramps	No. Accidents[a]	MV[b]	Accident Rate[c]	No. Ramps	No. Accidents[a]	MV[b]	Accident Rate[c]
Diamond Ramps	53	44	124.9	0.35	45	67	99.4	0.67
Trumpet Ramps	9	22	28.7	0.77	7	21	24.6	0.85
Cloverleaf Ramps w/o Collec. Dist.	48	83	111.2	0.75	59	135	155.8	0.87
Cloverleaf Ramps with Collec. Dist.	15	37	73.3	0.50	16	56	82.0	0.68
Cloverleaf Loops w/o Collec. Dist.	46	64	84.2	0.76	34	59	70.7	0.83
Cloverleaf Loops with Collec. Dist.	9	14	36.3	0.39	10	19	36.5	0.52
Left Side Ramps	5	14	18.9	0.74	11	81	46.4	1.74
Direct Connections	14	55	101.2	0.54	11	53	61.5	0.86
TOTAL[d]	**264**	**418**	**708.6**	**0.59**	**268**	**629**	**710.3**	**0.89**

NOTE: If the crossroad crosses under the freeway (mainline), the ramps are associated with an undercrossing. If the crossroad crosses over the freeway (mainline), the ramps are associated with an overcrossing. Overcrossing on-ramps are generally downgrades and off-ramps are generally upgrades. Undercrossing on-ramps are generally upgrades and off-ramps are generally downgrades.

[a] No. of Accidents.
[b] Million Vehicles.
[c] Accidents Per Million Vehicles.
[d] Total includes other ramp types studied.

TABLE 5 Ramp Accident Rates by Ramp Type, Undercrossing (2)

Type of Ramp	ON				OFF			
	No. Ramps	No. Accident[a]	MV[b]	Accident Rate[c]	No. Ramps	No. Accident[a]	MV[b]	Accident Rate[c]
Diamond Ramps	32	44	95.4	0.46	44	73	109.8	0.66
Trumpet Ramps	2	5	3.5	1.43	0	--	--	--
Cloverleaf Ramps w/o Collec. Dist.	27	72	105.4	0.68	19	86	76.0	1.13
Cloverleaf Ramps with Collec. Dist.	5	2	14.3	0.14	5	3	13.0	0.23
Cloverleaf Loops w/o Collec. Dist.	17	44	53.7	0.82	19	47	50.0	0.94
Cloverleaf Loops with Collec. Dist.	5	3	8.0	0.38	5	1	13.2	0.08
Left Side Ramps	2	11	8.0	1.38	4	124	47.0	2.64
Direct Connections	2	10	28.6	0.35	2	30	29.9	1.00
TOTAL	**92**	**191**	**316.9**	**0.60**	**98**	**364**	**338.9**	**1.07**

NOTE: If the crossroad crosses under the freeway (mainline), the ramps are associated with an undercrossing. If the crossroad crosses over the freeway (mainline), the ramps are associated with an overcrossing. Overcrossing on-ramps are generally downgrades and off-ramps are generally upgrades. Undercrossing on-ramps are generally upgrades and off-ramps are generally downgrades.

[a] No. of Accidents.
[b] Million Vehicles.
[c] Accidents Per Million Vehicles.

TABLE 6 Accident Rates by Type of Freeway Ramp (*2*)

Ramp Type	On	Off	On & Off
Diamond Ramps	0.40	0.67	0.53
Cloverleaf Ramps with Coll-Dist Roads[a]	0.45	0.62	0.61
Direct Connections	0.50	0.91	0.67
Cloverleaf Loops with Coll-Dist Roads[a]	0.38	0.40	0.69
Buttonhook Ramps	0.64	0.96	0.80
Loops with Coll-Dist Roads	0.78	0.88	0.83
Cloverleaf Ramps w/o Coll-Dist Roads	0.72	0.95	0.84
Trumpet Ramps	0.84	0.85	0.85
Scissor Ramps[b]	0.88	1.48	1.28
Left Side Ramps	0.93	2.19	1.91
Average	0.59	0.95	0.79

NOTE: Accident rates are per million vehicles.
[a]Only the On & Off rate includes the accidents occurring on the collector-distributor roads.
[b]A ramp that has opposing traffic crossing the ramp traffic under stop sign control.

TABLE 7 Accident Rates by Interchange Unit and Area Type (*4*)

RURAL

Interchange Unit	Vehicle Miles (100 Mil.)	No. Accidents[a]	Accident Rate[b]
Deceleration lane	2.51	348	137
Exit Ramp	0.57	199	346
Area between speed change lanes	6.52	554	85
Entrance Ramp	0.59	95	161
Acceleration lane	3.68	280	76
Acceleration - deceleration lane	0.49	87	116
Total	14.36	1,563	109[c]

URBAN

Interchange Unit	Vehicle Miles (100 Mil.)	No. Accidents[a]	Accident Rate[b]
Deceleration lane	5.83	1,089	186
Exit Ramp	1.48	546	370
Area between speed change lanes	11.87	1,982	167
Entrance Ramp	1.61	1,159	719
Acceleration lane	8.40	1,461	174
Acceleration - deceleration lane	2.45	555	227
Total	31.64	6,792	214[c]

[a]No. of Accidents.
[b]Accidents per 100 Million Vehicle-Miles.
[c]Average Accident Rate.

In summary, studies conclude that the design of cloverleaf ramps, scissor ramps, and left-side ramps should be avoided where possible. Collector-distributor roads should be considered in high-volume interchange designs and especially in designs for which loop and cloverleaf ramps are used.

Interchange Areas

Interchange areas include the areas along the freeway mainline between and including acceleration lanes, deceleration lanes, and their respetive ramps.

Accident rates in interchange areas are presented in Table 7 by interchange unit and area type (*4*).

Urban interchanges have much higher accident rates than rural interchanges. The exceptionally high rate of accidents on urban entrance ramps may be a result of the inadequate acceleration lanes found on many urban interstates. The relative safety of entrance and exit terminals is enhanced with geometric designs that provide 800-ft or longer acceleration or auxiliary lanes.

Deceleration lanes 900 ft or longer reduce traffic friction on the through lanes and account for reduced accident rates. Geometric designs for weaving maneuvers should provide weaving sections that are at least 800 ft long.

Based on the results of interchange operational studies, the potential for accidents has been related to the volume of the ramp traffic and the relationship between the ramp and through-lane traffic volumes (*5*). A general conclusion is that it is safer to merge or diverge a given volume of vehicles with or from a freeway at several minor flow ramps than at single high-volume on- and off-ramps.

Interchange Systems

As more interchange areas operate at or near capacity, the likelihood of increased speed differentials between upstream freeway sections and interchange sections increases.

Interchange capacity relative to interchange spacing was addressed by Cirillo (*4*). No definitive correlation between capacity and safety was found, aside from the direct relationship of volume increase and accident frequency. She did find, however, that accident rates increase when speeds vary from the mean speed of the freeway section.

As shown in Table 8, accident rates have been shown to increase as interchange spacing decreased in urban areas. Conversely, in rural areas, the change in rates was less dramatic. The effect of the spacing of urban interchanges on accident rates is an important design consideration because of greater frequency of interchanges as a result of increased traffic demand.

Interchange Improvements

Many older interchanges on the nation's highway system are reaching the end of their design lives and must be redesigned or rehabilitated. Safety improvements are an important consideration in interchange rehabilitation.

TABLE 8 Accident Rates by Proximity to Interchange Ahead or Behind (*4*)

EXIT SIDE

Dist. to exit-ramp nose ahead

Urban	No. Acc.[a]	Acc. Rate [b]
Less than .2 miles	722	131
.2-.4 miles	1,209	127
.5-.9 miles	786	110
1.0-1.9 miles	280	75
2.0-3.9 miles	166	63
4.0-7.9 miles	19	69
More than 8 miles[c]	--	--
Rural		
Less than .2 miles	160	76
.2-.4 miles	459	75
.5-.9 miles	559	69
1.0-1.9 miles	479	69
2.0-3.9 miles	222	68
4.0-7.9 miles	46	62
More than 8 miles[c]	--	--

ENTRANCE SIDE

Dist. to exit-ramp nose ahead

Urban	No. Acc.[a]	Acc. Rate [b]
Less than .2 miles	426	122
.2-.4 miles	1,156	125
.5-.9 miles	1,655	105
1.0-1.9 miles	278	84
2.0-3.9 miles	151	59
4.0-7.9 miles	200	75
More than 8 miles[c]	--	--
Rural		
Less than .2 miles	117	80
.2-.4 miles	482	82
.5-.9 miles	560	72
1.0-1.9 miles	435	64
2.0-3.9 miles	169	51
4.0-7.9 miles	52	40
More than 8 miles[c]	--	--

[a] No. of Accidents.
[b] Accidents per 100 Million Vehicle-Miles.
[c] No data available.

Evaluation of the effects of 37 interchange rehabilitatio projects on traffic safety was documented in one recent stud in which before and after accident rates were observed unde control conditions (*6*).

The results of this safety analysis revealed a statisticall significant reduction in accident rates for 13 projects, signif icant increases in accident rates for 2 projects, and no signif icant change in accident rates for 22 projects.

Table 9 presents the reduction in accident rates with dif ferent types of interchange rehabilitation. Modification refer to the element and level of improvement of the interchange modified during the rehabilitation projects. Modification t full diamonds may include lengthening of acceleration an deceleration lanes, adding ramp lanes, and optimizing existin or installing new traffic signals. Partial and full clover leaf improvements may include the addition of collecto distributor roads, lengthening of weave areas, and length ening of acceleration and deceleration lanes. The combine results from interchanges in each category are presented i Table 9. Study results led to the conclusion that interchang

TABLE 9 Before and After Safety Comparison of Interchange Rehabilitation Projects (*6*)

Modification	Observed Percent Reduction in Accident Rate	Statistical Significance @ 95% Confidence Level
Full Diamonds		
Major Geometric	20.7	No
Minor Ramp	32.0	Yes
Minor Crossroad	33.1	Yes
Minor Ramp & Crossroad	21.2	Yes
Full Cloverleafs		
Major Geometric	-11.5[b]	No
Minor Ramp & Collector-Distributor Rd	-55.8[b]	No
Minor Ramp & Crossroad	-7.8[b]	No
Partial Cloverleaf		
Major Geometric	38.4	Yes
Minor Ramp & Crossroad	45.5	Yes
Other Interchange Configurations		
Minor Ramp & Crossroad	8.2	No
Summary By Project Type		
Major Geometric	23.7	Yes
Minor Ramp & Crossroad	16.3	Yes
All Projects	18.7	Yes

[a] Accidents per Million Vehicles.
[b] Signifies an increase in Acc. Rate

rehabilitation projects are effective in reducing the number of accidents.

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REFERENCES

1. J. G. Yates. Relationship Between Curvature and Accident Experience on Loop and Outer Connection Ramps. In *Highway Research Record 312*, HRB, National Research Council, Washington, D.C., 1970, pp. 64–75.
2. R. A. Lundy. The Effect of Ramp Type and Geometry on Accidents. In *Highway Research Record 163*, HRB, National Research Council, Washington, D.C., 1967, pp. 80–117.
3. R. Ervin, M. Barnes, C. MacAdam, and R. Scott. *Impact of Special Geometric Features on Truck Operations at Interchanges.* Transportation Research Institute, University of Michigan, Ann Arbor, 1985, pp. 1–126.
4. J. A. Cirillo. Interstate System Accident Research Study II, Interim Report, Part I. In *Highway Research Record 188*, HRB, National Research Council, Washington, D.C., 1967, pp. 1–7; Part II, *Public Roads*, Washington, D.C., Aug. 1968, pp. 71–75.
5. *Traffic Control and Roadway Elements—Their Relationship to Highway Safety/Revised, Interchanges.* Highway Users Federation for Safety and Mobility, Washington, D.C., 1970, pp. 1–11.
6. D. W. Harwood and J. L. Graham. Rehabilitation of Existing Freeway-Arterial Highway Interchanges. In *Transportation Research Record 923*, TRB, National Research Council, Washington, D.C., 1983, pp. 18–25.

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Operational Considerations for Systems of Interchanges

JOEL P. LEISCH

Basic three-dimensional considerations in design include the composition of the elements of the highway and its effects on driver operation from a dynamic viewpoint. This discussion is an extension of basic three-dimensional design and covers interchange operational requirements, route considerations, and related signing. The features discussed are not as direct and are perhaps much more subtle than those that have to do with the geometrics of longitudinal and cross-sectional elements. Design considerations under this heading deal mostly with communicative aspects between the driver and the freeway and interchange complex in which the main thrust is to clarify, simplify, and facilitate driver operations. There are 13 operational and design criteria associated with freeway and interchange design. They are basic lanes, lane balance, applications of auxiliary lanes, route continuity, appropriate interchange form, no weaving within interchange on freeway, right exits and entrances only, single exit on freeway per interchange, exit in advance of crossroad, simplified signing, implementation of decision sight distance, freeway and exit ramp speed relationships, and ramp spacing. Although these operational and design criteria are discussed in various chapters of the 1990 AASHTO *Policy on Geometric Design of Highways and Streets*, the focus here is to clarify their application in freeway and interchange planning and design. Many of the concepts were first developed in the late 1950s and early 1960s, yet most were not incorporated in the AASHTO design policy until 1984—some 25 years after inception. The paper is intended for use as a practitioner's checklist of the 13 essential criteria during planning and designing a new freeway facility or considering operational and design improvements to an existing facility.

Basic three-dimensional considerations in design include the composition of the elements of the highway and its effects on driver operation from a dynamic viewpoint. The discussion in this paper is an extension of basic three-dimensional design and covers interchange operational requirements, route considerations, and related signing.

The features referred to here are not as direct and are perhaps much more subtle than those that have to do with the geometrics of longitudinal and cross-sectional elements. Design considerations under this heading deal mostly with communicative aspects between the driver and the freeway and interchange complex in which the main thrust is to clarify, simplify, regulate, and facilitate driver operations.

The operational and design considerations discussed here have evolved as a result of the experience gained and research accomplished during the past 60 years since the first interchange and controlled-access facility was constructed. Much of this has been as a result of human factors research associated with driver characteristics and expectations related to highway geometrics and traffic control devices.

Since it is the geometrics of the freeway and interchange system that dictates safe and efficient operations, an expanded definition of geometric design might be as follows: A dynamic facet of highway and traffic engineering, which, in its proper application, is a highly sophisticated and specialized discipline. It translates research and operational experience three-dimensionally into a physical highway plant, considering driver comfort and convenience, safety, and operational efficiency.

This expanded definition provides the purpose and gives direction to the designer. In following this definition there is the ability to look for new and better solutions in design of highways. These solutions should be tempered with results of operational experience, and the design should be approached from the viewpoint of all drivers—the stranger, the regular user, the angry, the harassed—who may be expected to use the facility as conceived by the designer. Should we not, then, provide a facility that allows the driver to perform tasks with a minimum of worry, indecision, and frustration?

If so, the driver should be able to see and know how to proceed along the highway. The task should be made so that it is easy to perform properly and difficult to perform improperly. However, should poor judgment be used or a mistake made, the highway should be "forgiving" and not exac[t] too great a price for a moment's inattention or indecision. The driver's attention should be drawn to what should be done and not to what should not be done. Transportation designers should simplify the driver's task and not complicate what already is complex for the driver operationally.

OPERATIONAL DESIGN FEATURES AND ROUTE CONSIDERATIONS

The research accomplished and experience gained in oper[ating] freeways has led to the establishment of operationa[l] design criteria that are vital in effecting safe and efficien[t] freeway operation consistent with the definition of geometri[c] design presented previously. Although application of appro[priate] dimensions, longitudinal and cross-sectional, is critica[l] in the planning and design process, of further consideratio[n] are the system aspects and communicative features that ten[d] to clarify and simplify operations through a uniformity i[n] design that satisfies driver expectancy.

During the late 1950s and 1960s, a series of operation[al] design criteria was formulated and documented (*1–4*). It wa[s] not until the 1984 AASHTO design policy was published th[at] most of these criteria actually became criteria. Many of th

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early articles and publications documenting these criteria were written by Jack Leisch. These publications were not referenced in either the 1984 or the 1990 AASHTO policies (*5*). The purpose of this paper is to recognize the Father of the criteria and to further clarify their importance in safe and efficient traffic operations.

The 13 criteria can be grouped into four concept categories:

- System criteria,
- Interchange considerations,
- Operational uniformity criteria, and
- Related or ancillary guidelines.

The categories will be discussed in detail in the following sections.

SYSTEM CRITERIA

When implemented, the system criteria, which include basic number of lanes, route continuity, lane balance, and application of auxiliary lanes, permit the freeway facility to operate in a sufficiently flexible mode to accommodate variations in volume and pattern of traffic. It is thus a design component to achieve a smoother flow of traffic to develop a more nearly uniform level of service, with improvement in driver comfort and convenience.

The first step toward determining the number of lanes required for flexibility in operation entails a capacity (level-of-service) analysis, predicated on normal peak hours, which are repeated daily during the morning (home-to-work) and the evening (work-to-home) periods; the capacity analysis is further extended to include any other known peaks, such as holiday or weekend concentration. This serves as a base on which appropriate number and arrangement of lanes are ultimately developed, with allowance for flexibility. The remaining steps in the process involve determination of the basic number of lanes, provision of lane balance, and application of special auxiliary lanes.

Basic Number of Lanes

Fundamental to establishing the number and arrangement of lanes on a freeway is the designation of the basic number of lanes. Consistency should be maintained in the number of lanes along an arterial facility. Thus, the basic number of lanes is defined as the minimum number of lanes designated and maintained over a significant length of the route, irrespective of changes in traffic volume and requirements for lane balance (Figure 1). In other words, it is a constant number of lanes assigned to a route, exclusive of auxiliary lanes. The number of lanes is predicated on the general volume of traffic over a substantial length of the facility. The volume considered here is the design hourly volume (normally representative of the morning or afternoon/evening weekday peak). Localized variations are ignored so that the volumes on individual segments between ramp terminals that are below the general level would theoretically have reserve capacity, whereas volumes on segments somewhat above the general level would be compensated for by the addition of auxiliary lanes introduced within these segments.

Required changes in the number of basic lanes are generally accomplished at major junctions (e.g., at freeway-to-freeway or system interchanges). In the case of an increase in basic lanes, the added lane is introduced via a 2-lane entering ramp at the system interchange. In the case of a decrease in basic lanes, the lane normally is not dropped at the ramp of the system interchange discharging the heavy volume, but via an exit at the following interchange. Another case in which the basic number of lanes may be reduced occurs when a series of exits, as in an outlying area of a city, causes the traffic load on the freeway to drop sufficiently to justify the smaller basic number of lanes. The selection of the basic number of lanes should be a matter of planning and design policy consistent with the overall system of freeways in a particular area.

Lane Balance and Auxiliary Lanes

Capacity analyses sometimes indicate abrupt changes in number of lanes at points of entrance or exit. Whereas such changes may be logical in terms of volume-capacity relations, they are not always appropriate in achieving smooth operating characteristics. To ensure efficient operation and to realize the indicated capacity potential where merging, diverging, and weaving take place, a certain balance of lanes must be maintained. Lane balance should comply with the relations shown in Figure 2.

The equations indicate that at exits the number of lanes approaching should be equal to one lane less than the combined number departing. At entrances, the combined number of lanes after the merge should either be equal to or one lane less than the total number of lanes approaching the merge. The principle of having an extra lane at the point of divergence (i.e., one more lane "going away") is a type of escape hatch, or a device that tends to flush traffic away from the point of divergence because of greater exit than approach capacity.

Since lane balance in effect produces lane drops on the freeway at certain exits, whereas basic lane arrangement

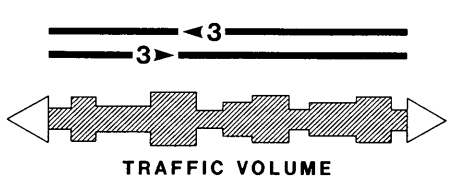


FIGURE 1 Basic number of lanes is maintained over a significant length of route.

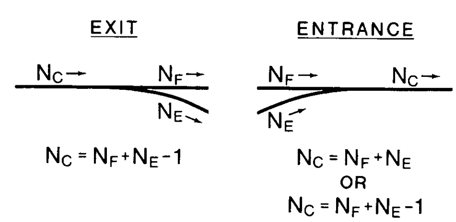


FIGURE 2 Lane balance (reduced lane changing).

maintains a constant number along the freeway, there appears to be a conflict between the two. This need not be so. The necessary requirements for maintaining both lane balance and basic lanes can be met by holding the basic number of lanes and then achieving lane balance by building on the basic number of lanes, that is, by adding auxiliary lanes or removing auxiliary lanes from the basic width of the traveled way. Thus, in no case would there be less than the basic number of lanes on the freeway.

To further illustrate the two situations, Figure 3 shows how lane balance and application of auxiliary lanes can be coordinated to produce desired arrangement. The two examples shown in Figure 3, Case A and Case B, demonstrate application of auxiliary lanes and establishment of lane balance to provide operational flexibility. Case A is to accommodate increases in entering and exiting traffic and resultant weaving between adjacent interchanges; Case B is to accommodate a volume increase over two or more interchanges requiring an auxiliary lane over a longer distance.

Any application of auxiliary lanes for the purpose described above must include consideration of an effective distance before the exit or beyond the entrance. Where interchanges are closely spaced and the auxiliary lane must be introduced at an entrance, the added lane should be carried to the exit of the following interchange or an added lane required for an exit should be extended back to the entrance of the previous interchange. An entrance followed by an exit frequently forms a weaving section, which requires the use of added width and certain minimum length (entrance to exit) to comply with capacity requirements for a weaving section. Here, an effective length of auxiliary lane on a full freeway should be of the order of 2,000 ft, preferably more, and should in no case be less than 1,500 ft. These controls govern where weaving capacity requirements alone may show lesser acceptable distances. On a facility serving as an adjunct to a freeway such as a collector-distributor road or a freeway distributor, a normal minimum length of auxiliary lane between an entrance and an exit is 1,000 ft.

Where interchanges are widely spaced, it might not be feasible or necessary to extend the auxiliary lane from one interchange to the next. In such cases, the auxiliary lane picked up at a two-lane entrance should be carried along the freeway for an effective distance beyond the merging point, or an auxiliary lane introduced on a two-lane exit should be carried along the freeway for an effective distance in advance of the exit and extended onto the ramp. Experience indicates that minimum distances of about 2,500 ft are needed to produce the necessary operational effect and to develop the full capacity of two-lane entrances and exits on high-type facilities.

Auxiliary lanes are essential to provide balanced and efficient operation. The objective is to add and remove auxiliary lanes on the freeway as required to account for localized increases and decreases in traffic volumes and to achieve a more uniform level of service. An auxiliary lane, however, has potential for trapping a driver at its termination point or where it is continued onto a ramp or turning roadway. Consequently, the driver should be made aware when traveling in or adjacent to an auxiliary lane. A special marking, contrasted with normal lane lines, and overhead signing should be provided for this purpose, and both should be compatible with the *Manual on Uniform Traffic Control Devices*.

The message conveyed by the marking becomes quite obvious to drivers. First, the basic lanes, those continuing through on the facility, are delineated, advising the through driver to stay to the left of the marking. Second, the marking informs the exiting driver to assume a position on the right of the marking. Third, it alerts the entering driver that it is necessary to cross over this marking to continue on the highway. The principle advanced here constitutes the means of providing the driver with prior information coupled with a visual indicator for positive guidance.

Route Continuity

Route continuity refers to the provision of a directional path along and throughout the length of a designated route. The designation pertains to a route number or to the name of a freeway.

Route continuity is an extension of operational uniformity coupled with the application of proper lane balance and the principle underlying the use of a basic number of lanes. Its attributes are of particular value to the unfamiliar driver who must rely on uniformity of design when presented with a choice of route. The uniformity associated with route continuity allows the driver approaching a bifurcation to be positioned properly across the lanes, followed by a confirmation received from route marking and directional signing.

In the process of keeping the driver on line, particularly in and around metropolitan areas, interchange configuration

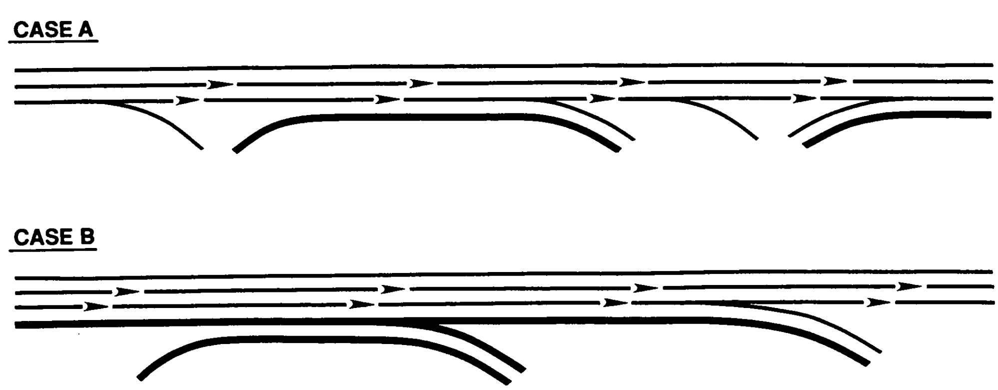


FIGURE 3 Auxiliary lane application.

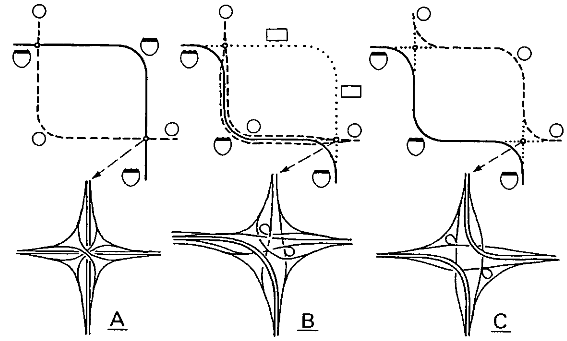


FIGURE 4 Route continuity.

must not necessarily favor the heavier movement at the point of bifurcation. It is the through facility (the designated route) that should always maintain its directional character. However, any predominant movement separating from the freeway should form a well-aligned exit on the right, equivalent operationally to the through movement.

Figure 4 shows the principle of route continuity as applied to a series of route configurations. It is important that the driver who wants to remain on route stays to the left and the driver who wants to leave or exit the route moves to the right and exits right.

INTERCHANGE CONSIDERATIONS

Two criteria are associated with interchanges: implementation of appropriate interchange form and no weaving within the interchange on the mainline of the freeway. The first relates to the correct interchange form and details of its design geometrics for the conditions at the location. The second relates to internal operations of the interchange to minimize vehicle conflicts and to facilitate safe and efficient operations.

Appropriate Interchange Form

The appropriate interchange form at any particular location is dictated by a variety of considerations, which may include the following:

- Classification of intersecting facilities,
- Volume and pattern of existing and future traffic,
- Physical constraints and right-of-way considerations,
- Environmental requirements,
- Local access and circulation considerations,
- Construction and maintenance costs, and
- Road user costs.

The interchange selection process can be greatly simplified through an understanding of the general characteristics of the various interchange forms. These general characteristics include capacity, safety, operations, right-of-way requirements, and construction costs. At any one interchange, there are perhaps only two or three forms that may fit the conditions (required characteristics). The appropriate interchange form(s) can initially be selected for further study based on the type or classification of the facility with which the freeway will

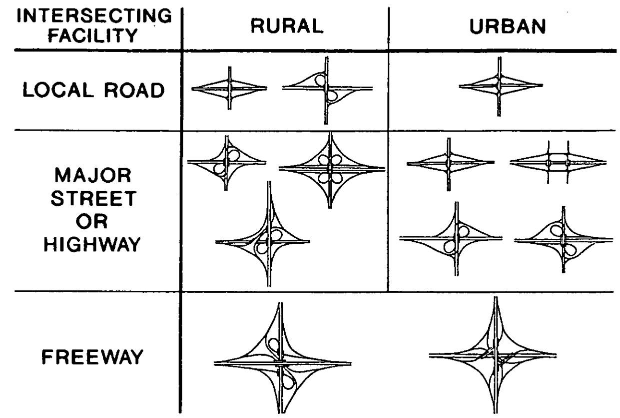


FIGURE 5 Adaptability of interchanges.

interchange. The matrix in Figure 5 associates basic interchange forms with type of interchanging facility in urban and rural areas. This is based on general operational, capacity, and right-of-way characteristics associated with the urban or rural location and facility type. It is only intended as a guide in beginning the interchange selection process.

Geometric variations of the basic forms that may be appropriate are not shown in Figure 5. An example may be in an urban area where a major or arterial street interchanges with the freeway. Other diamond interchange variations may be appropriate (e.g., three-level diamond, single point urban diamond, or compressed diamond). Figure 5 shows basic forms only and is intended as a guide.

No Weaving Within Interchange

Interchange forms as cloverleafs and some partial cloverleafs (loops in adjacent quadrants) result in weaving operations in the interchange. Such interchanges exhibit high accident experience and poor operational characteristics that usually affect not only entering and exiting traffic but mainline flow as well. To avoid such problems, collector-distributor roads can be added or the interchange can be converted to another form.

Figure 6 shows in concept the example solutions suggested here. In this case, a cloverleaf is modified or converted to

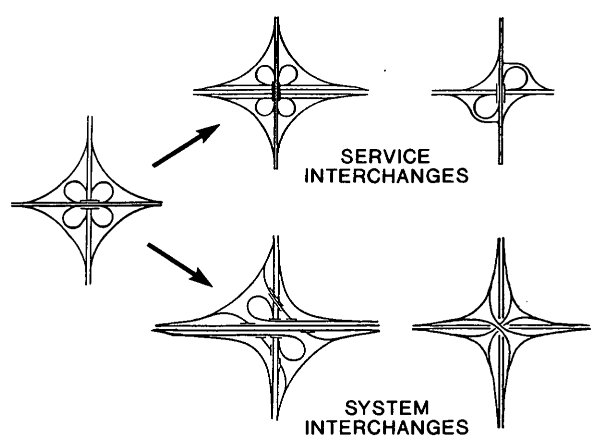


FIGURE 6 No interchange weaving.

other forms, depending on whether the interchange is a system interchange (freeway to freeway) or a service interchange (freeway to street).

OPERATION UNIFORMITY CRITERIA

When implemented, operation uniformity criteria produce a uniformity of operation along the freeway by facilitating and simplifying the driver's task, which results in more efficient operations.

Right Exits and Entrances

Much has been written about this criterion. It only needs to be emphasized here that right exits and entrances only should exist on a designated freeway route. This satisfies driver expectancy and keeps slow-moving vehicles from left lanes and avoids weaving across all lanes of the freeway. It should be noted that the accident rate at left-side ramps is twice that at right-side ramps.

Single Exit Per Interchange in Advance of Crossroad

This criterion is a critical one in simplifying the driver's task by providing only one decision point on the freeway and giving the driver a view of the exit ramp well in advance. Operational Uniformity can thus be achieved by implementing the previous criterion. Use of these criteria produces a uniform arrangement of exits and entrances along a freeway, providing for a uniform pattern of directional signing and allowing drivers to exit in a consistent manner at all interchanges, as shown in Figure 7.

The two freeway and interchange systems shown in the figure produce different operational characteristics, although the basic forms of the interchanges are identical. In the upper facility, a difficult or confusing pattern of exits is shown. Each interchange produces different operational characteristics along the freeway. Some interchanges result in two exits, some one exit. One interchange has one exit beyond the crossroad hidden from the driver's view, and the cloverleaf has not only two exits with one beyond the crossroad but a weaving section between the entering and exit loop ramps.

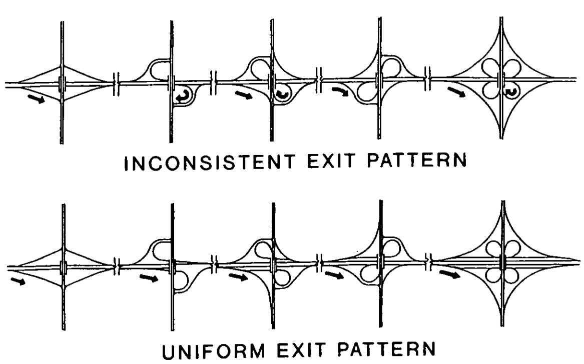


FIGURE 7 Operational uniformity.

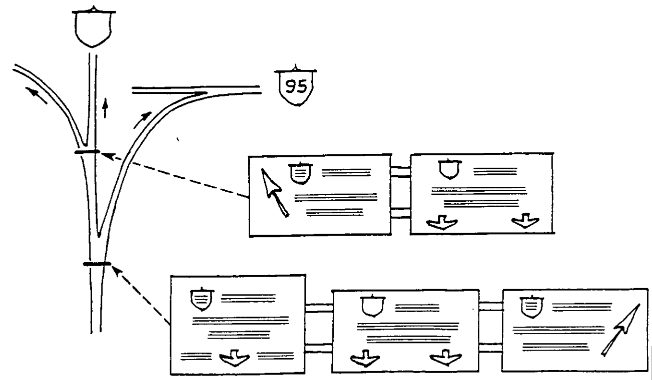


FIGURE 8 Complex signing.

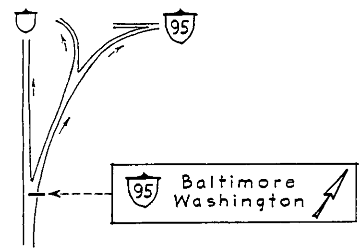


FIGURE 9 Simplified signing.

The facility in the lower portion of the figure with similar interchange forms has a uniform pattern of exits from the freeway. With geometric adjustments, each interchange has a single right exit in advance of the crossroad. This uniform pattern of exits also produces uniformity in signing along the freeway, further simplifying the driver's task.

Simplified Signing

This operational uniformity criterion is a result of the previous criteria. To demonstrate how signing can be simplified, two examples are shown in Figures 8 and 9. Figure 8 shows an interchange with two exits—one right and one left. The required sign panels and message units are shown to provide the information necessary to the driver to successfully negotiate the interchange. This can be compared with the single right exit design in advance of the crossroad portrayed in Figure 9. In this case, only one sign panel with four message units is necessary at the exit from the freeway—one decision point and no confusion. Once the driver exits and is operating at a lower speed, the ramp splits, in this case to go east or west. Supplemental signing at the ramp bifurcation would be provided to guide the driver to the desired destination.

ANCILLARY GUIDELINES

For lack of better term, the last three criteria are categorized as ancillary criteria. They are decision sight distance, freeway and ramp speed relationships, and ramp sequencing or spacing requirements.

Decision Sight Distance

This element or criterion in freeway design and operations relates to the distance at which a driver can perceive a decision point along the freeway. In most cases, these decision points are exits or lane drops. Major bridges and tunnels should also be considered because they may require significant driver adjustment to changing conditions. The criteria for determining decision sight distance are defined in Figure 10. The AASHTO policy clarifies the definition and longitudinal dimensions for various facilities and circumstances. The distances indicated in the figure for freeways with design speeds of 60 or 70 mph are within the range of values in the AASHTO policy. These distances of design speed are those necessary for the driver to perceive the decision point, react, and perform the appropriate maneuver.

Freeway and Exit Ramp Speed Relationships

This criterion refers to the distance required for the driver to decelerate the vehicle from the speed of the freeway to the speed of the controlling curve of the ramp (Figure 11). The dimensions indicated are from the physical gore of the exit ramp to the beginning of the controlling curve of the ramp. The assumption is that the vehicle is travelling at approximately the speed of the freeway at the gore and decelerates at a comfortable rate to the ramp curve (the dimensions have been rounded). Although the roadway element of length L in the figure is shown as a tangent, it may be a flat curve, a series of transition curves, or a spiral.

The purpose of providing this distance is not only for safe vehicle deceleration and operation but also to eliminate the need for drivers to decelerate on the freeway to safely negotiate the exit. This encourages more uniform speeds on the freeway and thus safer operation.

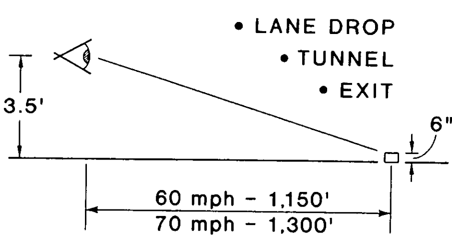
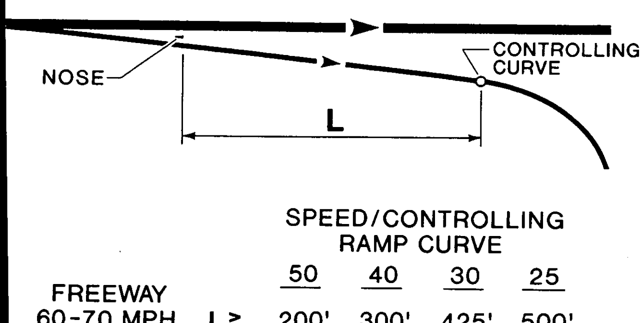


FIGURE 10 Decision sight distance.



IGURE 11 Freeway and exit ramp speed relationship.

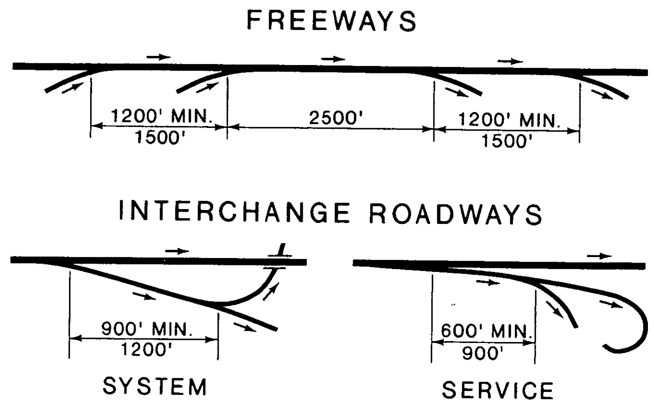


FIGURE 12 Ramp spacing.

Ramp Sequence

Dimensions for sequencing of ramps are described in the AASHTO policy on the basis of design requirements and, to an extent, capacity relationships. Some of the dimensions for successive exits or entrance, entrance followed by an exit, and ramp exits for system and service interchanges are shown in Figure 12. These dimensions, which are based on experience, have proved to be appropriate not only to accommodate ramp exit or entrance geometric criteria but also to take into account driver operational needs in spreading conflict or decision points. This also results in smoother freeway operations with more uniform operating speeds.

SUMMARY

Although the operational and design criteria discussed in this paper are discussed in various chapters of the 1990 AASHTO policy (*5*), the intention here is to clarify their application in freeway and interchange planning and design. Many of the concepts were first developed in the late 1950s and early 1960s, yet most were not incorporated in the AASHTO design policy until 1984—some 25 years after inception. This paper is also intended as a practitioner's checklist of the 13 essential criteria for planning and designing a new freeway facility or considering operational and design improvements to an existing facility.

REFERENCES

1. *Dynamic Design for Safety*. Institute of Traffic Engineers, Washington, D.C., 1972.
2. J. E. Leisch. Adaptability of Interchanges to Interstate Highways. *Transportation Journal*, ASCE, Jan. 1958.
3. R. M. Michaels. Human Factors in Highway Safety. *Traffic Quarterly*, 1961.
4. J. E. Leisch. Lane Determination Techniques for Freeway Facilities. *Proc., Canadian Good Roads*, 1965.
5. *A Policy on Geometric Design of Highways and Streets*. AASHTO, Washington, D.C., 1990.

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Geometric Design Features of Single-Point Urban Interchanges

DAVID R. MERRITT

The single-point urban interchange (SPUI) is a relatively new type of diamond interchange. It offers improved traffic-carrying ability, safer operation, and reduced right-of-way needs under certain conditions when compared with other interchange configurations. The distinguishing feature of this interchange is the convergence of all through and left-turning movements into a single, signalized intersection area. The more important visibility issues and geometric design features of the SPUI are summarized. The geometric considerations for left-turn path design and for placement of traffic control devices are discussed, and the effect of certain design features on bridge length is examined. The theme is that the complexity of the SPUI design requires careful selection of all design feature dimensions and an awareness of the impacts of design decisions on traffic operations and structural costs.

In most urban areas, traffic demand on existing arterial roadways has grown to the point where congestion is commonplace. The greatest restriction to traffic flow along these roadways is at signalized intersections. Common solutions to congestion problems at intersections include the addition of traffic lanes or, when right-of-way is unavailable, the grade separation of one or more through movements.

When grade separation is being considered, designers have traditionally considered conventional diamond and partial cloverleaf interchange configurations. One particular type of diamond interchange, sometimes called a compressed or tight urban diamond interchange (TUDI), has proven to be an efficient interchange configuration in terms of both minimal right-of-way requirements and operational performance. The TUDI design includes two closely spaced ramp and crossroad intersections that depend on a well-coordinated signal phasing arrangement for efficient traffic operation.

A new type of interchange has emerged recently (see Figure 1), commonly called the single-point urban interchange (SPUI). SPUI offers improved traffic-carrying ability, safer operation, and reduced right-of-way needs under certain conditions when compared with such other interchange configurations as TUDI and normal diamond-type interchanges (*1–4*). The distinguishing feature of this interchange is the convergence of all through and left-turn movements into one signalized intersection area on the crossroad versus two separate intersection areas. The advantage of this feature is that all intersecting movements can be served by a single signal with, at most, one stop required to pass through the interchange. Approximately 40 SPUIs are in operation in the United States, and a similar number are under construction or consideration.

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There are two basic types of SPUIs: those with the major road elevated over the ramp and crossroad intersection (i.e., an overpass SPUI, shown in Figure 1) and those with the major road depressed under the intersection area (i.e., an underpass SPUI, shown in Figure 2). Most SPUIs are overpasses.

The difference between the two SPUI types is most evident in the design of the bridge structure. The overpass SPUI typically includes a conventional, long single-span bridge; sometimes a three-span bridge is used. In contrast, the underpass SPUI usually requires a deck or platform-type structure to support the various intersecting traffic movements. These differences can be further illustrated by the length, depth, and number of spans in the bridge structure. A typical overpass design would have a single-span bridge of 220 ft in length and a depth of 8 to 9 ft. An underpass design would have two spans of about 70 ft in length and a depth of 3 to 4 ft.

Some important visibility issues and geometric design features of SPUI are summarized here, with particular emphasis on left-turn path design, placement of traffic control devices, and factors affecting bridge length.

VISIBILITY ISSUES

In any type of interchange, unobstructed visibility of the sign- ing, geometric configuration, pavement markings, signaliza- tion, and channelization features are necessary for the drive to safely and efficiently perform the correct maneuvers at th proper time while traveling through the interchange. Thi need for driver visibility at a SPUI is emphasized because left turn movements in a SPUI are quite different from those a the more common TUDI. Drivers who are unfamiliar wit the SPUI design may encounter some initial difficulty in per forming a left-turn maneuver from the crossroad or the off ramp. For this reason, the applicable sight distances and vis ibility lines of sight should be maximized.

Visibility Along the Major Road and Ramps

Visibility and line-of-sight considerations from the major roa to the exit-ramp signing and geometry are important desig considerations. The guide signing should preferably be ove head and conform to the *Manual on Uniform Traffic Contr Devices* (MUTCD) (*5*). Decision sight distance along the ma jor road approach to the exit ramp, as described in *A Poli on Geometric Design of Highways and Streets* (*6*, p. 12

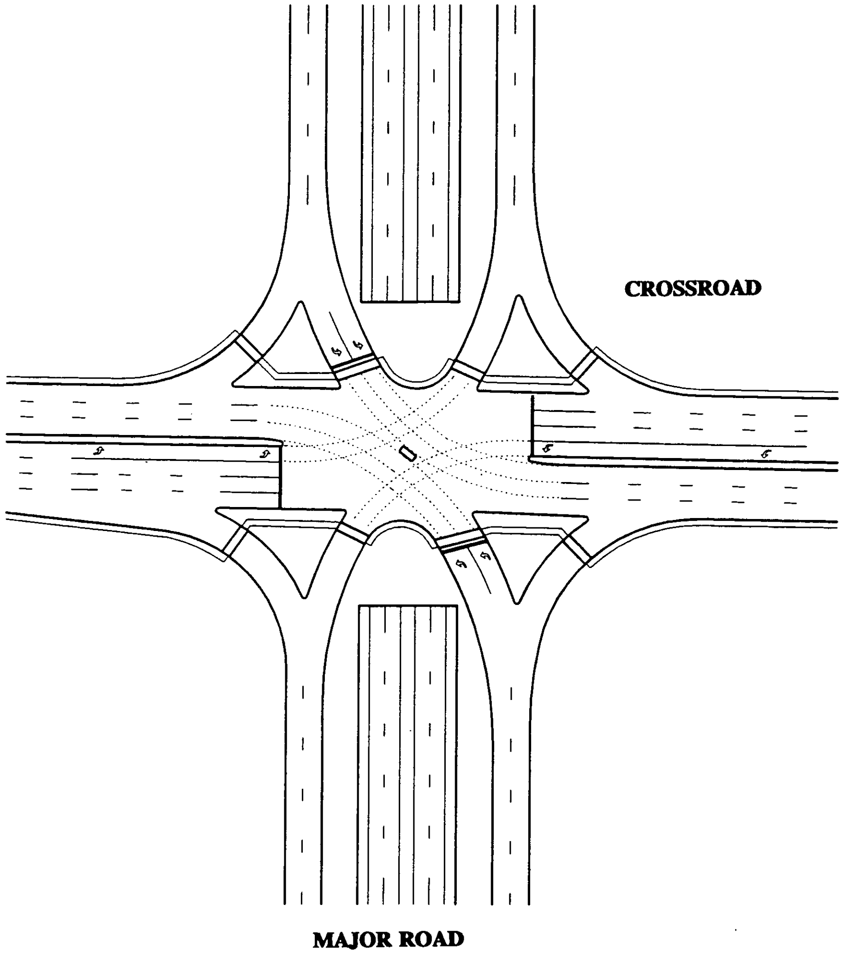

FIGURE 1 Overpass SPUI.

(commonly referred to as the Green Book), should be used as the minimum signing design criteria. Sight distances of 1,000 to 1,450 ft would be appropriate for major road speeds of 60 to 70 mph, depending on the location of the interchange (e.g., urban or rural).

The exit-ramp driver's visibility to the crossroad intersection is especially critical at a SPUI because the decision point to turn left or right will occur somewhat sooner at a SPUI than at other diamond-type interchanges. As a minimum, the horizontal and vertical exit-ramp alignment should provide desirable design values of stopping sight distance (*6*, pp. 284, 293). However, alignments based on decision sight distances (*6*, p. 125) should be used whenever possible. Extremely sharp horizontal or vertical curves should be avoided on the exit-ramps in the vicinity of the left- or right-turn decision point.

The point of initial driver perception of the large triangular intersection island and the point for the left- or right-turn decision should occur at or just beyond the gore point on the off-ramp from the major roadway. The target value of this island can be enhanced by installing appropriate warning and target delineation signs at the decision point and on the nose of the intersection island.

Visibility Along the Crossroad

Although not as critical as the major road, elements of the crossroad approaches to the SPUI should also provide for maximum driver visibility. Decision sight distance for the crossroad approach to the SPUI should be used as the minimum signing design criteria. As a minimum, crossroad horizontal and vertical alignment should provide desirable design values of stopping sight distance (*6*, pp. 284, 293). However, the crossroad alignment should provide intersection sight distances (*6*, p. 760) whenever possible.

Due to the SPUI's relatively unusual design, crossroad drivers rely heavily on guide signing, pavement markings, and lane use signing for the necessary positive guidance to travel safely through the SPUI intersection area. Sight triangle distances and sight lines to the traffic signals and other traffic control devices should be checked for conformance with the MUTCD (*5*, p. 4B-11). The presence of a small, mountable center island with signing and object markers will encourage the correct and proper flow of traffic through the area. High-quality pavement markings along the left-turn path also contribute positively to visibility for the left-turning driver.

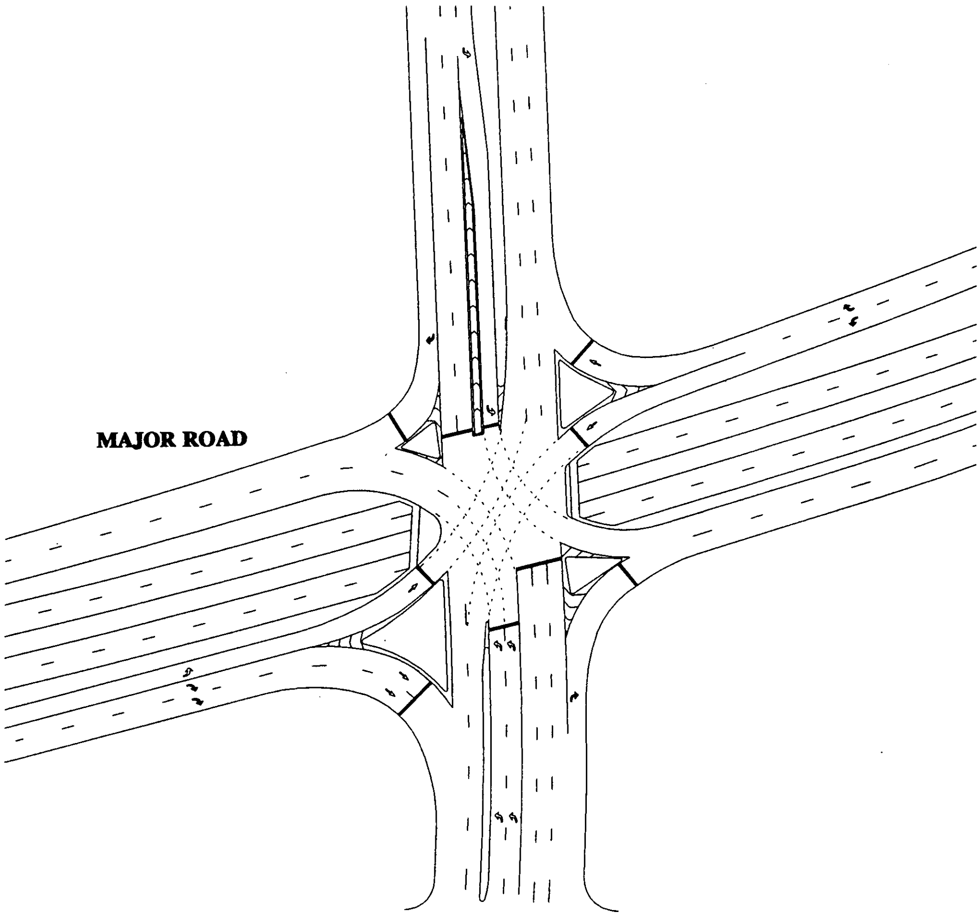

FIGURE 2 Underpass SPUI.

Another enhancement to driver visibility through an overpass SPUI is the use of a multispan structure instead of a single-span bridge. A longer, multispan bridge will provide a more open and less restrictive feeling to the driver as he or she approaches the intersection area. This openness increases the off-ramp left-turn driver's view of the crossroad and probably helps reduce the anxiety of all drivers traveling through the intersection area. A multispan bridge may alleviate driver anxiety by eliminating the "dark hole," or "tunnel," effect that is created by the SPUI's relatively wide overpass structure. Understructure lighting reduces this "dark hole" effect.

GEOMETRIC CRITERIA

Addressed in this section are design considerations for the SPUI left-turn roadway, which is one of the more unusual geometric features. Also examined are the impact of selected geometric features on the length of the bridge and the size of the intersection area.

Left-Turning Roadway Layout

Radius and Superelevation Rate

One of the SPUI's most unique features is its relatively high-speed left-turning roadways. The design of these roadways is complicated by the fact that a portion of the roadway is in the intersection area, and a portion is on the ramp. The portion in the intersection area should not have any superelevation, whereas the portion on the ramp should have a minimum slope for drainage purposes. The AASHTO Green Book (*6*, p. 197) provides some guidance on the relationship between design speed and radius for superelevated turning roadways.

The relationship between design speed and radius described in the AASHTO Green Book is as follows:

$$e + f = V^2/15R \qquad (1)$$

where

V = design speed (mph),
R = minimum radius of curvature (ft),
e = superelevation rate, and
f = side friction factor.

The results of several studies of turn speed (conducted in the early 1940s) are reported in the AASHTO Green Book to illustrate the relationship between design speed (i.e., 95th percentile speed) and side friction in Equation 1. Because none of these past studies specified the superelevation rate found at the study sites, representative rates were assumed by the authors of the AASHTO Green Book. The findings from these studies are summarized in the Green Book (*6*, p. 197) and in Table 1 of this paper.

The author is familiar with a number of well-designed SPUIs, and the radii of the left-turning roadways range generally from about 170 to 400 ft. These radii seem to be the most practical and reasonable for design purposes.

Superelevation is difficult to develop on any intersection approach (especially if there is a significant grade or skew in the intersecting alignments) because of the problems of transitioning from normal crown to superelevated to level section within a relatively short length of roadway.

The left-turning ramp roadway approaching the intersection area should be based on a nominal 0.02 ft^2 superelevation or a reverse crown. First, a nominal superelevation of 0.02 ft^2 will facilitate efficient drainage along the curbed, ramp portion of the turning roadway. Second, the added superelevation will allow drivers to travel at slightly higher speeds on the ramp than on the turn path within the intersection area. This will provide a comfortable speed transition along the ramp between the low-speed crossroad and the high-speed major road.

In summary, the radius of the left-turning roadway should be taken from the second column of Table 1 for a predetermined left-turn design speed.

Sight Distance

Left-turn drivers at SPUIs need stopping sight distance along the turning paths as well as intersection sight distance in the intersection area. This latter sight distance represents the minimum distance that a stopped vehicle must have of the conflicting traffic stream such that there is sufficient time to safely enter or cross the intersection from a stopped position. Because these distances are longer than stopping sight distances, designs based on providing stopping sight distance along the roadway do not guarantee adequate intersection sight distance.

In Figure 3, the sight distance for the left-turning roadways for an overpass SPUI is shown where a bridge abutment or slope treatment could restrict the sight distance. In the underpass type, sight restrictions could be created by a concrete bridge parapet wall or other obstruction, such as pedestrian fences and the like.

As a result, intersection sight distance availability should be checked for all turn movements at both the overpass and underpass types of SPUIs. Intersection sight distance values are described in the AASHTO Green Book (*6*, p. 760).

Intersection sight distance adequacy is sensitive to the amount of curvature in the vertical alignment of the crossroad. This concern is particularly applicable to underpass SPUIs because the intersection area at this SPUI type is often located on a crest curve. The rates of curvature (i.e., K values) provided in the AASHTO Green Book (*6*, pp. 284, 293) are based on providing stopping sight distance only. If these AASHTO Green Book values are used in the design of the vertical alignment through the intersection area, adequate intersection sight distance may not be provided. The crossroad vertical alignment should provide desirable values of stopping sight distance, as a minimum, and intersection sight distance whenever possible.

Stopping sight distance along the SPUI left-turn paths is also an important design consideration. This sight distance, shown in Figure 3, is actually across the inside of the curve and is dependent on the lateral location of sight obstructions (e.g., a bridge retaining wall or safety-shape barrier wall). As a result, the sight distance needs of the left-turn drivers on the SPUI can dictate the location of the bridge supports and, consequently, the length of the bridge.

A procedure for calculating the lateral clearance measured from the centerline of the left-most lane of the left-turning roadway to the furthest offset driver sight line is described in the AASHTO Green Book (*6*, p. 219). Using this procedure,

TABLE 1 Minimum Lateral Clearance to Obstruction

DESIGN SPEED (mph)	MINIMUM CURVATURE (ft)	SIGHT DISTANCE (ft)	LATERAL CLEARANCE (ft)
20	92	125	20.3
25	167	150	16.5
30	273	200	18.1
35	409	225	15.4
40	588	275	16.0

1. Maximum radius of curvature in feet for the design speed calculated from AASHTO Green Book for superelevation rate e=0.02 ft/ft
2. Minimum stopping sight distance from AASHTO Green Book Table III-1, pg. 120
3. Calculated lateral clearance distances from AASHTO Green Book Figure III-26A, pg 222

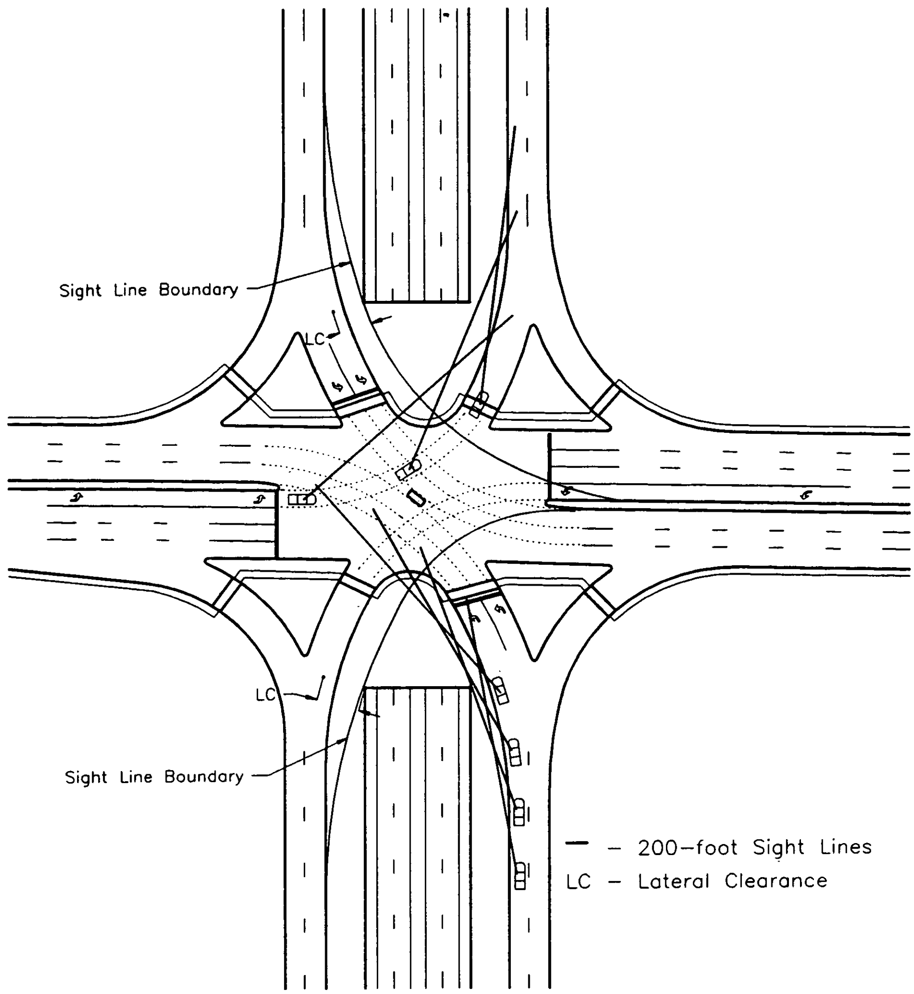

FIGURE 3 Sight distances on left-turning roadways.

the lateral clearance for the range of turn speeds and curvature commonly found at SPUIs will vary from 15 to 20 ft. Lateral clearances based on the left-turn radii are presented in the fourth column of Table 1.

As shown in Figure 3, the lateral clearance boundary formed by the intersection of all possible sight lines changes from zero a short distance before the curve, to a maximum value along most of the curve, and then back to zero following the curve. The lateral clearances in Table 1 represent the minimum values needed along most of the length of the curve. If the lateral clearance listed in Table 1 is provided, adequate stopping sight distance will be available everywhere along the ramp.

Design Speed

Generally, the choice of design speed for the left-turning roadways is critical to the safety and efficiency of the SPUI's operation. It can also have a significant effect on the cost of the structure. Efficient operations can be achieved with a minimum design speed of 25 mph and desirable values ranging from 30 to 35 mph. In cases in which there is a sharp alignment skew angle in alignments, a significant grade on the crossroad, adverse superelevation on the crossroad, or a combination of those factors, a compromise in design speed may be necessar to achieve a reasonable left-turning roadway design.

Curve Geometry

The left-turning roadway layout should be designed to provid a long, constant-radius curve geometry. Broken-back or com pound curves tend to produce undesirable traffic operations including lower discharge rates, increased lane encroachmen by turning vehicles, and higher speed differentials caused b driver hesitancy. Safely traversing a constant-radius curve i significantly easier for the drivers of larger, longer, and wid vehicles.

Another consideration in the SPUI intersection layout the required separation of the opposing left-turn travel path The relatively high speeds that can be attained on these tur ing roadways cause a certain amount of driver apprehensio in meeting oncoming vehicles. This apprehension can lead increased erratic maneuvers, unnecessary lane changing, slow vehicle speeds, higher vehicular speed differentials, and r duced operational efficiency. Therefore, a minimum of 6 between the outside edge lines of opposing left-turning mov ments and a minimum 10-ft vehicle body clearance should be provided.

Number of Lanes and Lane Width for Turning Roadways

Traditional traffic-capacity methods should be used to determine the number of lanes needed in the SPUI design. Designers should always consider dual lanes for both the crossroad and off-ramp left-turn movements whenever possible. Structural problems may arise during attempts to retrofit an existing SPUI to conform to the geometric design standards when the original bridge design provided only for a single-lane turning path.

The pavement width on turning roadways is generally increased slightly to accommodate the off-tracking characteristics of large trucks on curves. The determination of the width of a turning roadway is presented in the AASHTO Green Book (*6*, p. 202). Lane widths under Case I provide for off-tracking in a single lane but not for passing stalled vehicles on the turn path. Thus, Case I is most applicable for the design width of a single-lane left-turn path through the SPUI intersection area. Case II lane widths are based on providing for off-tracking in a single-lane and for passing a stalled vehicle. Case II is applicable for designing the width of single-lane left-turn paths along the SPUI on- and off-ramps. Lane widths for Case III provide for two-lane operation with off-tracking. Case III is applicable to dual-lane left-turn path design both in the intersection area and along the on- and off-ramps.

Grades

Grades on the crossroad through the intersection area should be the minimum necessary for drainage purposes. Grades associated with the ascending off-ramp and descending on-ramp combination of the underpass SPUI can be slightly steeper than AASHTO Green Book recommendations for ramp sections (*6*, p. 964) because gravitational forces can be used to the ramp driver's advantage during deceleration on the off-ramp and acceleration on the on-ramp. Sight distance should always be considered in the design of the ramp grades, as discussed earlier.

Effects of Alignment Skew

The alignments of most intersections and interchanges do not intersect at exactly 90 degrees. There is usually slight skew angle. The same is true of SPUIs. Skew angle is defined as the rotation of the crossroad alignment relative to the major roadway alignment—a clockwise rotation of the crossroad from normal, which results in a positive skew angle.

Skew between the alignments of the major and crossroads can have an adverse effect on SPUI traffic operation because of increased clearance distances, decreased travel speeds on two of the left-turn paths (via shortened radii), and increased difficulty for off-ramp right-turn drivers to see along the crossroad. Alignment skew can also have an effect on the location of the bridge supports, which may require increasing the length of the bridge.

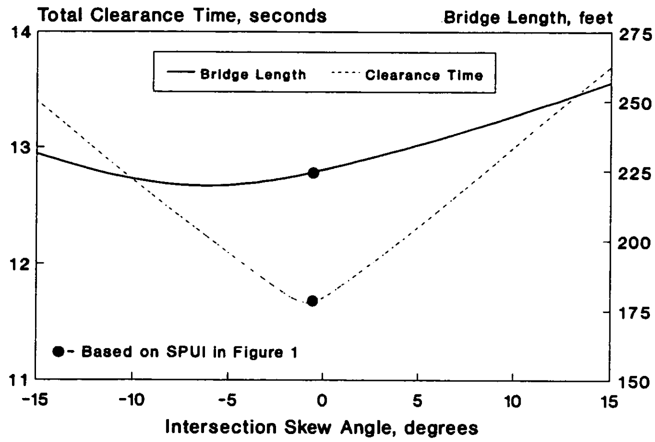

FIGURE 4 Effect of skew angle on clearance time and bridge length.

The effect of skew angle has been examined using a mathematical model of the geometric relationships between the ramp, crossroad, and bridge abutment locations based on ramp driver sight distance requirements (*7*).

Figure 4 illustrates the effect of skew angle on the total all-red clearance time and bridge length for the SPUI shown in Figure 1. Total all-red clearance time represents the sum of the three clearance intervals associated with the crossroad left-turn, crossroad through, and off-ramp left-turn signal phases. Because all-red time represents time not available to serve SPUI traffic, it is a useful measure of the impact of skew on the operational efficiency of the SPUI.

In summary, skew in the alignments can increase the clearance time and bridge length. Large skew angles should be avoided in the design of a SPUI.

Crossroad Median Considerations

Like skew angle, median width also has an effect on bridge length and all-red clearance time. The relationship between median width and bridge length is relatively straightforward. The width of the crossroad cross section must be increased to accommodate an increase in the width of any of its components. For example, if the median is widened, the bridge that spans it must be lengthened. This relationship is shown in Figure 5.

Also shown in Figure 5 is the effect of median width on all-red clearance time. This figure suggests that there is a nonzero median width that yields a minimum all-red clearance time of about 6 ft for this particular SPUI. This analysis assumes that the median nose has compound curvature such that the median curb is designed to conform to the radius of the off-ramp left-turn path until such a point that it reaches a nominal 4-ft width and can be "capped" with a 2-ft nose radius. This design technique is commonly used at most at-grade intersections with wide medians. Its benefit at the SPUI is that it allows the two crossroad median noses to be brought closer together. Since the median nose is typically used to locate the stop lines of the crossroad traffic movements, this technique effectively minimizes the size of the intersection area and the length of the associated clearance paths.

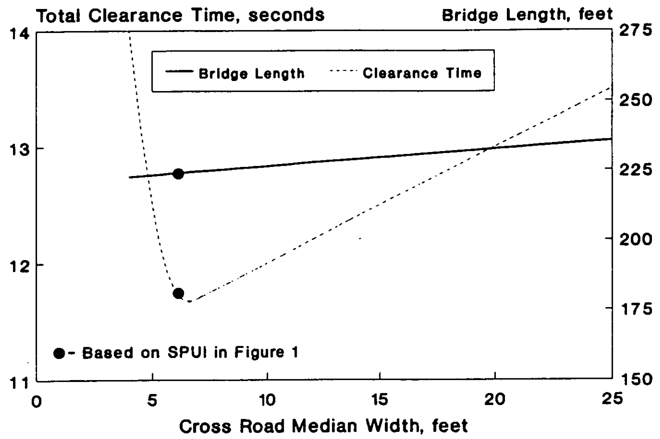

FIGURE 5 Effect of median width on clearance time and bridge length.

In summary, a median of nominal width combined with the median design technique described above can minimize the length of the all-red clearance time.

OTHER DESIGN CONSIDERATIONS

Pedestrian Accommodations

At typical at-grade intersections, pedestrians are accommodated within the signalization by provision of a coincident through-vehicle phase. Unfortunately, at a SPUI, pedestrians crossing the crossroad do not have a coincident through phase because this phase corresponds with the grade-separated major road movement (unless there are frontage road movements). As a result, an exclusive, actuated pedestrian phase is needed or pedestrian movement is not provided for. Pedestrians could be directed to the nearest intersection to safely cross the crossroad.

When a pedestrian phase is not included in the three-phase SPUI signalization, pedestrians crossing the crossroad will not be able to complete the crossing during one signal phase. Pedestrians will have to cross to the median and wait there until a subsequent phase allows them to cross to the other side. If this type of pedestrian crossing is provided, the crossroad median should be sufficiently wide to provide for a pedestrian refuge area.

When an actuated pedestrian phase is included in the three-phase SPUI signalization, the phase is typically assigned to operate concurrently with the adjacent off-ramp left-turn movement. This operation has the advantage of providing some concurrent vehicular traffic service, although it is limited to serving pedestrians on one side of the SPUI at a time. If pedestrian demands are sufficiently high such that they must be served on both sides of the SPUI, then an exclusive pedestrian phase may need to be considered. If pedestrian signalization is provided, the crossroad median would not need to be designed as a place of pedestrian refuge.

Proximity of Nearby Intersections

The distance to the closest intersection measured along the crossroad is a critical issue for all interchange configurations. The ability to provide safe and efficient left- and right-turn movements from the off-ramp to the crossroad at SPUIs is directly dependent on the location, spacing, and signal coordination of the adjacent intersections. If the signal controllers at the SPUI and adjacent intersections are not coordinated, there may be undesirable speed differentials and a high potential for rear-end type collisions. The results of two recent studies of accident patterns at SPUIs (*8,9*) indicate that rear-end accidents on the off-ramps are the predominant type of accident at SPUIs.

Efficient signal coordination at all signalized junctions typically requires a minimum intersection spacing to ensure efficient traffic progression over a range of travel speeds and cycle lengths. Stover et al. (*10*) studied the effect of intersection spacing on arterials and developed relationships among travel speed, spacing, and cycle length that maximize progression efficiency. The results of their study indicate that a ½-mi spacing between signalized intersections will yield the maximum progression efficiency for travel speeds between 35 and 40 mph and cycle lengths between 90 and 100 sec. Because SPUIs also usually require longer signal cycle lengths than conventional intersections, they will fit well into a progressive coordinated system. The ½-mi spacing is desirable between the SPUI and adjacent crossroad intersections. A spacing to adjacent intersections should be about 1,000 ft. This will provide sufficient distance for left-turn bay development and some limited traffic progression opportunities. Lesser spacing is usually the case in urban conditions, but longer distances should be considered during the interchange design process.

Access Control

Another SPUI design consideration is the control of access on the crossroad. Full access control for a reasonable distance from the ramp intersections is needed to provide for the higher speed turns that are common at SPUIs. The AASHTO Green Book (*6*, p. 841) indicates that driveways and entrance approaches should be prohibited along the crossroad within the SPUI's "functional boundary." On the approach side of the crossroad, this functional boundary is generally interpreted to extend a distance equal to or greater than the combined left-turn bay taper and storage length. On the departure side, the functional boundary should extend a reasonable distance beyond the entrance to the off-ramp.

Traffic Signal Placement

Traffic signal placement is an important design decision at a SPUI. Traffic signals at existing SPUIs have been installed over the center island, on the outside beams of the bridge structure, on span wire in advance of the structure, on combination overhead signal and sign structures, and on the triangular islands adjacent to the ramps. Generally, signal heads that are centered over the travel lanes they control offer more positive guidance to the drivers than do the heads mounted adjacent to the traveled way.

Signal heads are mounted either vertically or horizontally at SPUIs. Displays at most existing SPUIs are mounted vertically. When vertical heads are centered over the travel lanes at overpass SPUIs, they should be always external to the bridge structure. Vertical signal heads mounted on the outside girders of the bridge structure and centered over the travel lanes are the preferred design. If terrain and other conditions permit adequate vertical underclearance at little or no additional construction cost, horizontal signal heads suspended over the lanes beneath the structure may be desirable.

The visibility of the signal heads controlling the off-ramp left-turn movement is critical. An advance signal may be needed on the larger triangular island on the off-ramp approach to provide the driver advance notice of the signals ahead. A "pull through" signal on the opposite triangular island can also be placed at SPUIs where travel distances through the intersection are relatively long. In all cases, the placement of the signals, when viewed from the off-ramp approach, should conform to the horizontal and vertical sight-line criteria described in the MUTCD (*5*, p. 4B–11). For the overpass SPUI, he edge of the bridge abutment should be checked for conflict with the visibility of the off-ramp signal heads. The minimum visibility distance to a signal head mounted over the centerline of the off-ramp left-turn lane will normally be provided if the ateral clearances presented in Table 1 are used in the layout geometrics of the interchange.

igning, Pavement Markings, and Island Channelization

The geometric layout of SPUIs must be coordinated with the igning, pavement markings, and island channelization. Signng at SPUIs is similar to that of other diamond-type interhanges except larger legends and advance signing should be sed because of the higher turning speeds in the interchange. Approach directional signing should be mounted overhead vith a large legend both on the major roadway and the crossoad. The placement of the support systems for the overhead igning should be considered when developing the preliminary nterchange design geometrics.

Pavement markings at SPUIs are also similar to those at ther diamond interchanges except for those associated with igher speed left turns. The lane line markings for these moveents must be maintained at a higher level of visibility to rovide better positive guidance to drivers. In some cases, mbedded pavement marking lights, commonly used on airort runways, have been used to enhance the delineation of ft-turn movements. The impact of these lights on drivers as not been formally studied, but all indications suggest that e lights have more of a novelty effect than a true operational r safety benefit. When a signal is installed beneath the strucre, a small island with signing and delineation is commonly sed to protect the under-hanging signal array and to effecvely divide and separate the opposing left-turn movements. he shape and size of this small island are important design nsiderations. The island shape is governed by the travel th geometry in the intersection area and is normally in the ape of a small parallelogram or rectangle. The island size dependent on the island being large enough to encompass e signing and delineation treatment and satisfying the ASHTO Green Book (*6*, p. 722) recommendation that isnds in urban areas are a minimum of 50 ft^2, preferably larger. sed on observations of numerous SPUIs by the author, it is recommended that one side of the island should be around 10 ft long; the minimum recommended length is about 6 ft (*11*).

Right-of-Way Usage

One of the advantages of the SPUI design is the minimal crossroad right-of-way required as compared with other diamond interchange forms. TUDIs with low to moderate left-turn demands on the crossroad typically do not have advance left-turn lanes and may have a slightly narrower right-of-way. TUDIs with extremely high left-turn demands will probably require exclusive left-turn lanes and the right-of-way may be somewhat wider than the SPUI because the TUDI's signal operations will not allow these lanes to be overlapped.

The right-of-way required along the SPUI's major road is considerably narrower than most other interchange forms but is about the same as TUDI. The right-of-way requirements of the SPUI, the distance between the outside back-of-curbs of each on- and off-ramp pair generally ranges from 200 to 600 ft, with an average of about 300 ft. By comparison, TUDIs are generally designed for rights-of-way from 250 to 400 ft.

SUMMARY

All aspects of the SPUI design should be thoroughly examined during the preliminary design stage to ensure that it is both economically and operationally efficient. In no other interchange configuration are the details of the bridge design so interrelated to the geometric and traffic control device design features. Well-designed SPUIs emphasizing the principles of positive guidance can operate safely and efficiently under a wide range of traffic conditions. Every effort should be made to use desirable design values for all features of the SPUI design.

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REFERENCES

1. T. W. Hawkes and M. D. Falini. The Urban Interchange. *Public Works*, Feb. 1987, pp. 45–46.
2. S. J. Brown and G. Walters. The Single-Signal Interchange. *ITE 1988 Compendium of Technical Papers*, Institute of Transportation Engineers, Washington, D.C., 1988, pp. 180–184.

3. B. Martin. Urban Diamonds—Justified or Not? *Technical Views*, Vol. 1, No. 1, Howard, Needles, Tammen, & Bergendoff, New York, N.Y., Winter 1989, pp. 27–30.
4. J. M. Witkowski. Benefit Analysis for Urban Grade Separated Interchanges. *Journal of Transportation Engineering*, Vol. 114, No. 1, ASCE, New York, N.Y., Jan. 1988, pp. 93–109.
5. *Manual on Uniform Traffic Devices*. FHWA, U.S. Department of Transportation, Washington D.C., 1988.
6. *A Policy on Geometric Design of Highways and Streets*. AASHTO, Washington, D.C., 1990.
7. J. A. Bonneson. *Factors Affecting Bridge Size and Clearance Time of the Single-Point Urban Interchange*. University of Nebraska, Lincoln, Aug. 1991.
8. J. A. Bonneson and C. J. Messer. *A National Survey of Single-Point Urban Interchanges*. Report TTI-2-18-88-1148-1. Texas Transportation Institute, College Station, March 1988.
9. E. Cheng. Accident Analysis for Single Point Urban Interchange. *1991 Compendium of Technical Papers*, Institute of Transportation Engineers, Washington, D.C., pp. 29–32.
10. V. G. Stover, P. B. Demosthenes, and E. M. Weesner. Signalized Intersection Spacing: An Element of Access Management. *1991 Compendium of Technical Papers*, Institute of Transportation Engineers, Washington, D.C., pp. 176–181.
11. C. J. Messer, J. A. Bonneson, S. D. Anderson, and W. F. Farland. *NCHRP Report 345*: *Single Point Urban Interchange Design and Operations Analysis*. TRB, National Research Council, Washington, D.C., 1991.

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Ramp/Mainline Speed Relationships and Design Considerations

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

Great care must be exercised in selecting the design speed of ramps, particularly off-ramps from high-speed freeways, to ensure that drivers slow down from the mainline freeway speed to the design speed of the ramp. The AASHTO Green Book provides guidance on the selection of ramp design speeds that are appropriate for specified mainline highway design speeds. The process of selecting an appropriate design speed for a ramp, the operational and safety problems that may arise from selecting a ramp design speed that is too low, and the geometric design and traffic control techniques that may be used to alleviate such problems are addressed. Where problems are anticipated, geometric design changes may be appropriate to increase the ramp design speed or the deceleration distance available to drivers. Traffic control devices, including advisory speed signing, may be used to communicate to drivers the need to reduce speed. Further evaluation of the effectiveness of speed-control measures is needed.

One of the difficult challenges facing designers of freeway interchanges is the development of ramp geometrics that are consistent with the mainline freeway speeds and geometrics. If a ramp has horizontal curves with design speeds much lower than the mainline freeway design speed, operational and safety problems may be created for vehicles traversing the ramp. Designers would often like to use relatively high ramp design speeds, but ramp geometrics are often strongly influenced by physical constraints in the interchange area. It may be especially difficult in a rehabilitation situation to identify feasible methods to increase the radius of a ramp curve at reasonable cost without the need for major realignment of other interchange elements. Similar issues arise in the design of ramps on arterials and collectors, although arterials and collectors often have lower speeds than freeways. The issues inherent in selection of appropriate ramp design speeds and geometrics to avoid interchange operational and safety problems are reviewed.

GEOMETRIC DESIGN POLICIES FOR FREEWAY RAMPS

The AASHTO Green Book (*1*) presents guidelines for the selection of the design speed for a ramp as related to the design speed of the mainline highway. The guidelines are summarized in Table X-1 of the Green Book, which is reproduced as Table 1.

AASHTO policy states that ramp speeds should, desirably, approximate the low-volume running speeds of the intersecting highways. Since such high design speeds for ramps are not always practicable, lower design speeds may be necessary. Table 1 presents three speed ranges for ramp design speed—identified in the table as the upper, medium, and low ranges. The AASHTO policy also states that only the design speeds over 50 mph in Table 1 are applicable to freeway and expressway exits (i.e., off-ramps).

The upper-range values for ramp design speeds in Table 1 are generally 5 to 10 mph less than the design speed of the mainline highway. The middle-range values are generally 15 to 20 mph less than the mainline highway design speed. The lower-range values are generally 25 to 35 mph less than the mainline highway design speed. The AASHTO policy states that ramp design speeds should not be less than the lower-range design speeds given in Table 1.

The ramp design speeds in the table apply to the sharpest or controlling ramp curve, usually on the ramp proper. The geometrics of ramp curves—as a function of design speed and maximum superelevation rate—are determined in accordance with AASHTO policy that applies to all horizontal curves, presented in Chapter III of the Green Book. For freeway ramps, the horizontal curve design criteria should be determined by reference to Table III-6 of the Green Book (see the next section for a discussion of horizontal curve design on arterials and collectors). Some elements of ramp curve design, such as superelevation runoff distances, are governed by the design guidelines for turning roadways in Chapter IX of the Green Book.

The following guidelines are given in Chapter X of the Green Book for selection of design speeds on specific types of ramps:

- For ramps that serve right-turn movements, upper-range design speeds are often attainable and the lower range is usually practicable. For diamond interchange ramps, a design speed in the middle range is usually practical.
- For loop ramps that serve left-turn movements, the upper-range values are not attainable. Loop ramps with design speeds above 30 mph require large areas of land and are, therefore, more costly to construct and maintain and require left-turning drivers to travel increased distances. The large land area requirements may make loop ramps with high design speeds infeasible in developed areas.
- For semidirect connection ramps, ramp design speeds in the upper and middle ranges can generally be used. Design speeds less than 30 mph should not be used.

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TABLE 1 Guide Values for Ramp Design Speed as Related to Highway Design Speed (*1*)

Highway design speed (mph)	30	40	50	60	65	70
Ramp design speed (mph)						
Upper range	25	35	45	50	55	60
Middle range	20	30	35	45	45	50
Lower range	15	20	25	30	30	35
Corresponding minimum radius (ft)	See Table III-6					

- For direct connection ramps, ramp design speeds in the middle and upper ranges should also be used. Ramp design speeds should generally be 40 mph or more and should in no case be less than 35 mph.

Ramps that connect highways with different design speeds should be designed to provide a smooth speed transition between these highways. In general, the highway with the higher design speed should be the control in selecting the design speed for the ramp (i.e., the higher of the two highway design speeds should be used to enter Table 1). However, the transition to or from the highway with the lower design speed also needs to be considered. On an exit ramp from a high-speed freeway to a lower-speed arterial, the design speed of the ramp proper (including the sharpest or controlling curve) should be determined from the freeway design speed using Table 1. It may also be appropriate to use a lower design speed for the final curve before the arterial ramp terminal to assist in creating an appropriate speed transition. On an entrance ramp from a lower-speed arterial to a high-speed freeway, the first curve that the driver encounters on the ramp after leaving the arterial may be based on the arterial design speed. However, the design speed of most of the ramp—and especially the final curve before entering the freeway—should be determined from the freeway design speed using Table 1.

The design speeds in Table 1 do not apply to ramp terminals, which should be properly transitioned and provided with speed-change facilities adequate for the roadway speeds involved. Speed-change lanes are usually provided at free-flow ramp terminals. Speed-change lanes are classified in AASHTO policy as auxiliary lanes and are more commonly referred to as acceleration and deceleration lanes. Ramp terminals that do not have free-flow connections are referred to as at-grade ramp terminals. Such terminals function as at-grade intersections and are typically found at the junction of a ramp with an arterial or collector facility. A complete explanation of AASHTO policies concerning the design of at-grade ramp terminals is provided by Plummer et al. in this Record.

GEOMETRIC DESIGN CRITERIA FOR RAMPS ON ARTERIALS AND COLLECTORS

The ramp design speeds given in Table 1 for the upper, middle, and lower ranges of ramp design speed as a function of the mainline highway design speed are applicable to ramps on arterials and collectors as well as freeway ramps. For arterials and collectors, Table 1 is entered using the design speed of the arterial or collector as the highway design speed. Thus, the main body of Table 1 is not used any differently on arterials and collectors than on freeways, except that urban arterials and collectors are likely to involve design speeds below 50 mph, which are not used on freeways.

However, the final line of Table 1, which indicates the minimum radii for horizontal curve design, does not fully explain the AASHTO policy as it applies to the design of horizontal curves on arterial and collector ramps and turning roadways. The table implies that the horizontal curve design criteria in Table III-6 apply to all horizontal curves on ramps. Whereas this is clearly true for freeway ramps, the table should make clear that the AASHTO policy provides two sets of design criteria for horizontal curves on arterials and collectors: (*a*) the high-speed design criteria based on Table III-6 of the Green Book, which are used on freeways and higher-speed arterials and collectors, and (*b*) the low-speed design criteria based on Table III-17 of the Green Book, which are used on lower-speed arterials and collectors. The low-speed design criteria for horizontal curves are based on higher net lateral acceleration or *f*-values than the high-speed criteria. The "high-speed" and "low-speed" horizontal curve design criteria should not be confused with the high, middle, and low ranges of ramp design speeds in Table 1.

AASHTO policy states that the low-speed design criteria for horizontal curves are applicable to arterials with design speeds of 30 mph or less and collectors with design speeds of 40 mph or less. State highway agencies operate few roads of this type and those roads seldom have ramps. However, it is more common for local agencies to design ramps or turning roadways under these low-speed conditions. Since the AASHTO Green Book is used by many local agencies, it would be desirable if Table 1 (i.e., Table X-1 of the Green Book) made clear that the low-speed horizontal curve design criteria may be used on some ramps.

The text that accompanies Table X-1 (see pp. 965–966 of the 1990 Green Book) states that the maximum superelevation rates for ramps are those given in Table IX-12 of the Green Book. (As discussed above, this is an apparent contradiction of Table X-1.) The superelevation rates in Table IX-12 are based directly on the low-speed horizontal curve design criteria in Table III-17 of the Green Book. Thus, the Green Book appears to sanction the use of low-speed horizontal curve design for all horizontal curves on ramps for which the ramp design speed is 40 mph or less. However, the authors of this paper do not believe that this is what was intended.

Figure 1 shows what the authors believe was intended a AASHTO policy. The figure indicates that the choice betwee horizontal curve design criteria based on high-speed desig (Table III-6) and low-speed design (Table III-17) for a specifi ramp on a specific highway should be based on the highwa functional classification and highway design speed (not th ramp design speed). Once either the high- or low-speed desig criteria have been selected, the ramp design speed should b used in designing the horizontal curve in accordance wit either Table III-6 or Table III-17.

Figure 1 indicates that all horizontal curves on freeway ramp should be designed in accordance with the high-speed horizont curve design criteria in Table III-6 of the Green Book. F arterials, the low-speed design criteria would apply to situatio

Functional Classification	Highway Design Speed (mph)								
	30	35	40	45	50	55	60	65	70
Freeway or Expressway	NOT APPLICABLE				Use Table III-6				
Arterial	Use Table III-17	Minimum: Use Table III-17 Desirable: Use Table III-6			Use Table III-6				
Collector	Use Table III-17		Minimum: Use Table III-17 Desirable: Use Table III-6		Use Table III-6				

FIGURE 1 AASHTO horizontal curve design criteria for ramps as a function of the design speed and functional classification of the mainline highway.

in which the arterial design speed (and thus the ramp design speed) was 30 mph or less. The high-speed design criteria in Table III-6 would apply to arterials with design speeds of 50 mph or more. For arterials with highway design speeds of 35, 40, and 45 mph, the low-speed horizontal design criteria in Table III-17 would be minimum acceptable values, and the high-speed criteria in Table III-6 would be desirable values for ramp design. A similar interpretation would apply to ramps and turning roadways on collectors, except that low-speed design for collectors applies to highway design speeds of 40 mph or less.

In summary, Table X-1 of the Green Book and its accompanying discussion do not clearly state the AASHTO policy concerning the design of horizontal curves on ramps. The table and its accompanying text need clarification. Figure 1 is our attempt to present what we think was intended by AASHTO.

OPERATIONAL AND SAFETY CONSIDERATIONS IN SELECTING RAMP DESIGN SPEED

Clearly, ramp design speeds are often dictated by physical design constraints. Designers would often prefer to use ramp design speeds higher than those they find it necessary to use. The following discussion examines the situations in which reduced ramp design speeds may lead to operational or safety problems and possible methods of avoiding such problems.

Over the years, highway agencies have found some ramp curves with concentrations of run-off-road and rollover accidents, particularly accidents involving trucks. Such problems are more prevalent on off-ramps than on-ramps because of the higher travel speeds of vehicles exiting from a freeway; speeds of vehicles entering a freeway from the arterial street system are likely to be lower. Both operational experience and a recent analysis of horizontal curve design criteria suggest that these safety problems are more related to vehicles traveling faster than the design speed than to any basic flaw in the AASHTO horizontal curve design policy.

A recent analysis by Harwood et al. (*2*) assessed the AASHTO high-speed design policy for horizontal curves as presented in Table III-6 of the Green Book. The study concluded that the current AASHTO design policy provides an adequate margin of safety for both passenger cars and trucks on horizontal curves as long as the design assumptions on which the AASHTO policy is based are not violated. In particular, it is important that trucks not travel faster than the design speed on curves with relatively low design speeds. Harwood et al. concluded that the current AASHTO horizontal curve design policy was adequate for both passenger cars and trucks traveling at or below the design speed of the highway. However, in some cases, a vehicle with very poor tires on a poor wet pavement could skid, or a vehicle with a worst-case rollover threshold could roll over, at only a few miles per hour above the design speed. Furthermore, the research found that skidding or rollover was most likely to be critical for curves with lower design speeds, such as ramps.

Table 2 summarizes vehicle speeds at impending skid and impending rollover for several critical scenarios. The table represents the following conditions:

- A minimum-radius AASHTO curve with a maximum superelevation rate of 0.08 ft/ft designed in accordance with Table III-6 of the Green Book,
- Wet-pavement friction levels equivalent to those assumed in AASHTO stopping sight policy,
- A passenger car rollover threshold equal to 1.2 *g*, and
- A truck rollover threshold of 0.30 *g*, which is representative of the worst-case trucks currently on the road.

TABLE 2 Vehicle Speed at Impending Skid and Rollover Under Critical Conditions (*2*)

Design speed (mph)	Maximum e	Passenger car speed (mph): At impending skid (wet)	Passenger car speed (mph): At impending rollover	Truck speed (mph): At impending skid (wet)	Truck speed (mph): At impending rollover
20	0.08	32.5	45.3	26.8	24.7
30	0.08	47.1	69.6	39.0	37.9
40	0.08	61.8	94.8	51.3	51.6
50	0.08	76.8	121.1	63.9	66.0
60	0.08	95.2	152.2	79.3	82.9
70	0.08	118.0	191.5	98.5	104.3

Table 2 indicates that the most critical design conditions for horizontal curves occur at the lower design speeds, particularly at design speeds of 20 to 30 mph. Under the assumed conditions (an extremely rare combination of worst-case conditions), a truck on a curve with a 20-mph design speed can roll over when traveling at about 25 mph and may skid off the road under critical wet-pavement conditions at about 27 mph. A truck on a curve with a design speed of 30 mph could roll over when traveling at a speed of about 38 mph and could skid off the road at 30 mph. It is important to note in Table 2 that passenger cars are less critical than trucks and that higher design speeds are less critical than lower design speeds.

Table 2 indicates the importance of ensuring that ramps have realistic design speeds. In other words, there is no problem in using a 30-mph design speed on a ramp, but we must be able to ensure that drivers—and, in particular, truck drivers—travel at or below 30 mph. However, safety problems may arise if designs are overly optimistic about how much speed reduction can be expected from drivers on an exit ramp from a high-speed freeway.

The study by Harwood et al. (*2*) did not address the AASHTO low-speed design criteria for horizontal curves presented in Table III-17 of the Green Book. These criteria are used only on turning roadways at ramps or intersections on low-speed arterials and collectors. Two key factors distinguish the low- and high-speed designs. First, horizontal curves designed in accordance with the low-speed criteria are based on higher *f*-values, so they may be more critical for skidding or rollover, especially by trucks. On the other hand, such curves are found only on lower-speed roadways, where it is less likely that vehicles will attempt to traverse the ramp at speeds substantially higher than the design speed. The issue of safety of passenger cars and trucks on curves designed in accordance with Table III-17 of the Green Book merits further investigation to fully evaluate the implications of current policy.

CONSIDERATION OF EXPECTED OPERATING SPEED IN DESIGN

Design policy for horizontal curves should ensure an adequate margin of safety against both rollover and skidding at the travel speeds actually used by vehicles on a particular horizontal curve. In other words, it is not enough just to select a design speed for a ramp to fit the physical constraints of the site. There is also a need to determine an anticipated operating speed for the ramp. If a substantial percentage of vehicles are expected to travel faster than the design speed, there is a need to change to a higher design speed or to incorporate effective speed-control measures in the design. The following discussion presents guidelines for selecting an appropriate ramp design speed and incorporating speed-control measures where needed.

DESIGN AND OPERATIONAL GUIDELINES

The following guidelines should be considered in selecting the design speed for an off-ramp:

- Consider physical and economic constraints in selecting a tentative design speed for the ramp. Use the upper or middle range in Table 1, if possible. It is especially important to avoid the lower range of ramp design speeds on ramps that will carry substantial volumes of truck traffic.
- Identify the most critical curve on the ramp (usually, but not necessarily, the first curve downstream of the gore area).
- Develop a forecast of operating speeds at the most critical curve on the ramp on the basis of actual speeds on existing ramps with similar mainline design speeds, mainline operating speeds, and similar geometrics for the speed-change lane and the portion of the ramp prior to the most critical curve. This forecast should be based on the mainline design and operating speeds, but not on the ramp design speed.
- If the projected ramp operating speed exceeds the design speed, raise the design speed. If the design speed cannot be raised because of physical or economic constraints, consider speed-control measures such as those discussed below.

On ramps where anticipated operating speeds exceed the maximum feasible design speed, the following speed-control measures should be considered:

- Provide signing with an appropriate advisory speed for the ramp.
- Place the advisory speed signing so that drivers have sufficient length to slow down between the signing and the most critical curve.
- Increase the length of the deceleration lane or realign the ramp to increase the distance from the gore area to the most critical curve.
- Supplement the standard advisory speed signing to make the signing more conspicuous, to increase the distance from the signing to the most critical curve, and to draw the attention of truck drivers to the signing. These objectives may be accomplished by using more than one ramp speed advisory speed sign, placing ramp speed advisory signing on the mainline highway in advance of the ramp, incorporating an Exit Speed panel in the guide signing for the off-ramp, using overhead signing, using a Truck Speed advisory sign, and using flashing beacons to call attention to the advisory speed.
- Avoid designs in which the presence of a critical curve on a ramp is not obvious (e.g., where a tight horizontal curve follows a larger-radius curve).
- Consider the use of collector-distributor roads in the interchange. Collector-distributor roads introduce an intermediate speed roadway between the mainline freeway and the ramp and, thus, may assist in reducing ramp speeds. For example, collector-distributor roads could be appropriate if design constraints necessitated the use of a loop ramp with a 30-mph design speed on a 70-mph freeway.

All of these speed-control measures have been used by highway agencies, but only limited data are available to quantify their effectiveness in reducing speeds. Highway agencies have found that traffic control devices alone are not very effective in reducing vehicle speeds on ramps, but this is not well documented, and further research is needed.

SUMMARY AND CONCLUSIONS

Current AASHTO policies for freeway ramp design are adequate so long as the drivers adjust their speed to levels th are less than or equal to the design speed. Safety problems on ramp curves are most likely for vehicles traveling faster than the design speed. Under normal and even worst-case conditions, drivers who stay within the design speed are unlikely to lose control of their vehicles. However, loss of control—due to skidding tires or vehicle rollover—is more likely for vehicles that exceed the design speed. Loss of control due to skidding or rollover is more likely for trucks than for passenger cars and is more likely for ramps with lower design speeds than for higher design speeds. Further research is needed to address the potential for skidding and rollover problems on ramp curves designed in accordance with the AASHTO horizontal curve criteria for low-speed design (see Table III-17 of the Green Book).

Where ramp designers find it necessary to use a reduced design speed for a ramp (especially for design speeds within the lower design speed range in Table 1), an assessment should be made as to whether drivers are likely to slow down to the selected design speed. If operating speeds higher than the design speed are expected, the design speed of the ramp should be increased whenever possible. If no change in ramp geometrics is possible due to physical, environmental, or economic constraints (especially in a rehabilitation project at an existing interchange), appropriate speed-control measures should be considered. Further research is needed to evaluate the effectiveness of speed-control measures, such as various forms of advisory speed signing, in reducing the travel speeds of passenger cars and trucks on ramp curves and to develop more effective speed-control methods.

REFERENCES

1. *A Policy on Geometric Design of Highways and Streets.* American Association of State Highway and Transportation Officials, Washington, D.C., 1990.
2. D.W. Harwood, J. M. Mason, W. D. Glauz, B. T. Kulakowski, and K. Fitzpatrick. *Truck Characteristics for Use in Highway Design and Operation.* Reports FHWA-RD-89-226 and FHWA-RD-89-227. Federal Highway Administration, U.S. Department of Transportation, 1990.

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Ramp Exit/Entrance Design—Taper Versus Parallel and Critical Dimensions

FRANK J. KOEPKE

The basic design criteria, and thereby design standards, used by governmental agencies to design exit and entrance ramp terminals have not changed in more than 30 years. However, the design details of the ramp terminals vary across the country. A survey of state departments of transportation (DOTs) indicates that there is mixed use of (*a*) only tapered lanes, (*b*) only parallel lanes, and (*c*) a combination of both tapered- and parallel-lane design. Forty-one (91 percent) of 45 state DOTs that responded to a nationwide NCHRP survey prefer a tapered design for exit ramps. Thirty-four (75 percent) of the responding states use a parallel design for entrance ramps. Most agencies use AASHTO policies as a basis for speed-change lane design and either comply with or exceed AASHTO recommendations for deceleration lane lengths. However, some state standards indicate minimum lengths of acceleration lanes that are less than the minimum lengths recommended by AASHTO. Most research indicates that operational aspects of the current design elements are acceptable for today's driving conditions. The "gore" or "wedge" of exit ramps ranks high in the location of freeway accidents, and some problems exist with respect to driver gap acceptance on entrance ramps. Both conditions have been attributed to the assumption that drivers do not know how to properly use, or just do not properly use, speed-change lanes.

For many years, in fact for decades, considerable discussion has been given to the operational aspects of either the taper- or parallel-lane design of freeway ramp terminals—the speed-change lanes. Some speed-change lane design criteria are based on data collected as far back as the 1940s and 1950s.

Although design elements vary between a tapered and a parallel ramp terminal, operational aspects are normally not that different. The taper design works on the principle of direct entry to or exit from a freeway at a flat angle. The parallel design provides an added lane for speed-change purposes. In theory, the taper design reduces the amount of driver steering control and, especially on exit ramps, fits well the direct path preferred by most drivers. However, taper design used on entrance ramps requires the driver to time-share between the tasks of accelerating, searching for an acceptable gap, and steering along the lane.

The parallel-type design requires a reverse-curve maneuver when merging or diverging but provides the driver with full view from side or rear-view mirrors to monitor following traffic. Most research indicates that either type, when properly designed, will operate satisfactory. Figure 1, adapted from *A Policy on Geometric Design of Highways and Streets* (*1*), commonly called the Green Book, illustrates taper and parallel design for both exit and entrance ramps.

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CURRENT PRACTICES

NCHRP sent survey forms to 60 design agencies to determine the state of the art of ramp terminal design (*2*). Agencies included those of all 50 states, FHWA, and several large design firms. The survey form included questions regarding the type of speed-change lanes used, ramp terminal design criteria, and operational experiences. Forty-five responses were received and reviewed. A summary of survey responses indicate that 4 agencies (9 percent) preferred the use of parallel design, 11 (24 percent) preferred a taper design, and 30 (67 percent) used both parallel and taper design. Agencies using both parallel and taper designs use a taper for exit ramps and a parallel lane for entrance ramps.

Most agencies use AASHTO policies as a basis for speed-change lane design and either comply with or exceed AASHTO recommendations for deceleration lane lengths. There is some difference in minimum acceleration lane lengths, where some state standards are less than the minimum lengths recommended by AASHTO.

THE EXIT PROCESS

The operational areas of both speed-change lanes (accelera tion and deceleration) are operationally composed of thre sections. The deceleration or exit lane begins with a tape section, the section in which drivers laterally shift from th through lane to the deceleration lane. For parallel ramp de sign, this is the section that begins at the edge of the freewa lane and transitions to the full ramp lane width, normally 1 ft. For taper ramp design, this is the section from the freewa edge to a point along the taper that is 12 ft from the edge c the freeway pavement. For design purposes, it is assumed tha this maneuver is done in 3.5 sec and that no deceleration i conducted while in this section. Therefore, the taper sectio is not included as part of the speed-change length. The secon section is the length of lane in which drivers decelerate i gear without applying brakes, which averages an addition 3.0 sec. The third section is the length in which the driv decelerates by applying brakes until the speed is equal to th average running speed of the first ramp curve. The time ne essary to complete this maneuver depends on the radius c the first ramp curve beyond the ramp "gore" or "wedg area. For design purposes, the second and third sections a designed as one unit.

AASHTO and all but four states surveyed during the NCHR study prefer the taper design for exit ramps. In additio research by Davis (*3*) indicates that nearly all exiting vehic

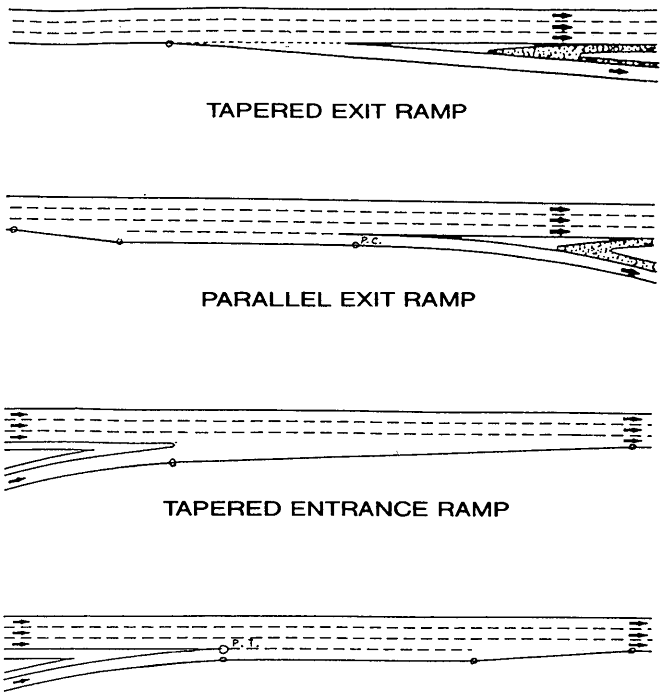


FIGURE 1 Typical ramp exit/entrance design (*1*).

(95 percent) tend to drive directly for the ramp proper even if the ramp terminal is designed as a parallel type. They did not use the taper and then drive parallel to the through lane. In fact, it was found that most vehicles began entering the deceleration lane in the taper section but did not completely clear the through lane until they were 50 to 200 ft from the ramp nose. Davis concluded that a tapered exit ramp would fit vehicle paths better than the parallel type and that AASHTO-recommended lengths for deceleration seem to be sufficient for the necessary speed reduction.

THE ENTRANCE PROCESS

The entry process differs from the exit process in that in addition to making a lane switch, the driver must make a gap search and acceptance decision. Traversing the entrance or acceleration lane involves maneuvering decisions from the initial approach to the speed-change lane, to acceleration, and then to the maneuver to enter the freeway though lane.

The first of the three operational sections of the entrance lane begins when the ramp driver transitions from the curvature of the ramp proper to the flatter geometry of the speed-change lane. It is recommended that the last curve of the ramp proper be designed for at least the average running speed of the highway. The second section allows for acceleration to the freeway running speed and the evaluation of gaps in freeway traffic. This section is the key component of the entrance lane. The third section is the taper area. The taper length for parallel-type ramps should be a minimum of 300 ft and for taper-type ramps should begin when the ramp width equals 12 ft. As with the exit ramp, the taper area is not included in the speed-change length. The slope, or taper ratio, of a taper-type ramp, of the ramp edge of pavement with respect to the freeway edge of pavement should remain the same from the ramp curve to the end of the entrance lane. A taper design with a 50:1 taper ratio is recommended as a desirable minimum.

Research by Polus and Livneh (*4*) indicates that drivers on parallel lines tend to merge before the middle of the acceleration lane. Drivers on tapered lanes tend to merge between the $\frac{1}{2}$ and $\frac{3}{4}$ section of the acceleration lane length. The research also indicates that acceleration rates are moderate to low compared with the capability of modern vehicles.

In general, states vary in the use of either taper or parallel ramp design. However, the Green Book (*1*) indicates "a decided trend toward the use of the taper type, both for deceleration and acceleration."

Most researchers viewed the operational problems associated with speed-change lanes as (*a*) drivers misunderstand the proper use of the lane, (*b*) the speed differential between ramp traffic and main line traffic is higher than the 5-mph speed difference used by AASHTO, and (*c*) drivers accept shorter taps in freeway traffic than has been expected.

Lundy (*5*) conducted a study of the effect of ramp type and geometry on accidents that indicates that accident rates on entrance ramps were consistently lower than exit ramp accident rates.

DESIGN CRITERIA

Factors considered in the design of speed-change lanes include the design speed of the highway, the design speed of the ramp curve adjacent to the speed-change lane, roadway gradients, and traffic volumes.

Exit Ramps

Figure 2 shows some of the design elements associated with exit ramps. One of the most critical elements is the distance provided to decelerate from at least the average running speed of the highway to the design speed of the initial ramp curve. The design speed of the ramp curve is dependent on the radius of the ramp curve and the rate of ramp superelevation. A vehicle should be able to leave the highway lane, transition laterally to the ramp proper, and decelerate to ramp speed, all within the "speed-change" portion of the exit terminal.

Table 1 indicates current AASHTO minimum deceleration lengths for exit terminals with grades of 2 percent or less. The table is reproduced from the 1990 Green Book (*1*) and has not changed since the 1965 AASHTO Blue Book (*6*). Lengths are based on a 3-sec time period for deceleration in gear.

As mentioned above, the taper section is not considered part of the speed-change length. AASHTO recommends that the taper section be 250 ft in length if the ramp is a parallel type and, if the design is a taper type, that the transition area be from the freeway edge of pavement to a point where the ramp pavement is 12 ft wide. Current AASHTO policy (*1*) recommends angles of divergence for taper-type ramps of between 2 and 5 degrees.

The exit "gore" is the area downstream from the point that the ramp left edge of pavement diverges from the freeway

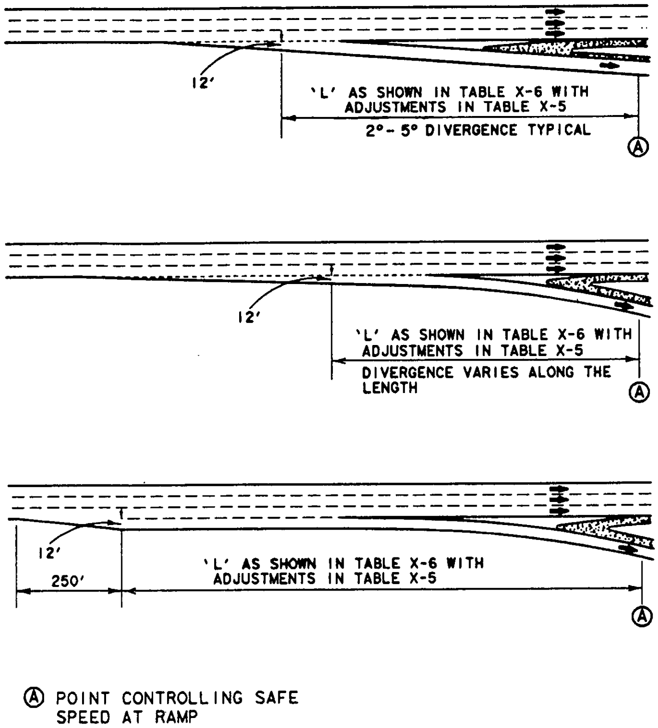


FIGURE 2 Exit ramps, single lane (*1*). *Top*, tapered design, tangent; *middle*, tapered design, curvilinear; *bottom*, parallel design.

edge of pavement to the gore nose. The gore nose is typically 20 to 30 ft wide and is the end of a paved neutral area. The entire area should be delineated with pavement markings. The entire section of the neutral area should be paved with either a combination of concrete pavement and shoulder material or with shoulder material. The unpaved area beyond the gore nose should be as level as possible. A typical gore area is shown in Figure 3.

A modification of the typical core area shown in Figure 4 is a gore that includes recovery areas. The accident rate at

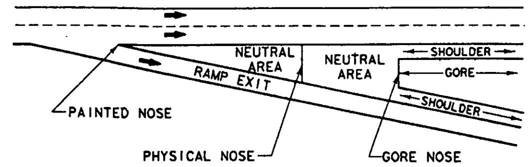


FIGURE 3 Typical gore area characteristics (*1*).

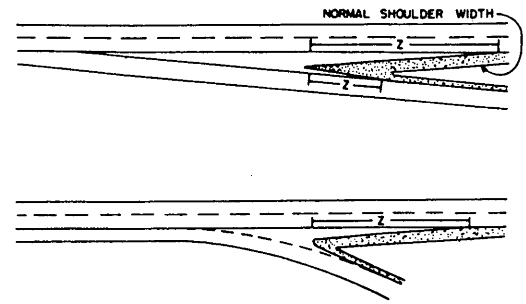


FIGURE 4 Exit gore with recovery area (*1*). *Top*, taper type; *bottom*, parallel type.

gore areas is one of the highest, and the inclusion of recovery areas will improve safety conditions. Figure 4 shows an exit gore with recovery areas along both the freeway and the ramp. It is recommended that the recovery area along the freeway be at least the width of the paved shoulder and that the recovery area adjacent to the ramp be a minimum of 3 ft wide.

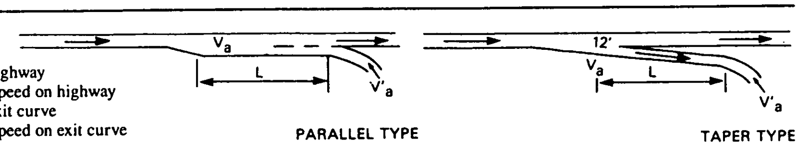
The recovery areas (Area Z in Figure 4) should taper to the respective edges of pavement. AASHTO recommends a taper ratio of one-half the design speed of the approach highway (i.e., a 70-mph freeway design speed would require a 35:1 recovery area taper, whereas a 50-mph ramp design speed would require a 25:1 recovery area taper).

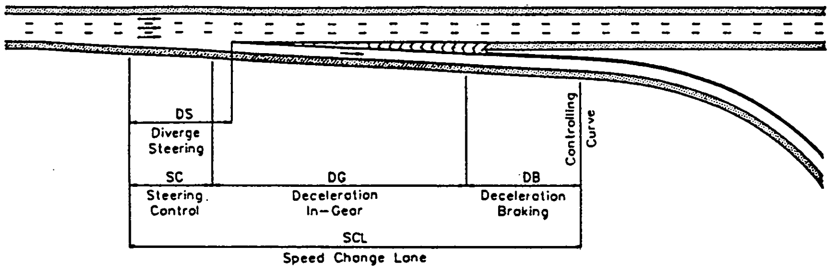
As part of NCHRP 3-35 (*2*), exit and entrance models were developed on the basis of driver behavior and traffic flow characteristics obtained from field studies and known human

TABLE 1 Minimum Deceleration Lengths for Exit Terminals with Flat Grades of 2 percent or Less

Highway Design Speed, V (mph)	Average Running Speed, V_a (mph)	Deceleration Length, L (ft) For Design Speed of Exit Curve, V' (mph)								
		Stop Condition	15	20	25	30	35	40	45	50
		For Average Running Speed on Exit Curve, V'_a (mph)								
		0	14	18	22	26	30	36	40	44
30	28	235	185	160	140	—	—	—	—	—
40	36	315	295	265	235	185	155	—	—	—
50	44	435	405	385	355	315	285	225	175	—
60	52	530	500	490	460	430	410	340	300	240
65	55	570	540	530	490	480	430	380	330	280
70	58	615	590	570	550	510	490	430	390	340

V = Design speed of highway
V_a = Average running speed on highway
V' = Design speed of exit curve
V'_a = Average running speed on exit curve





SC - Steering Control Zone in which the driver steers and positions a vehicle from the freeway lane onto the deceleration lane.

DS - Diverge Steering Zone which is the distance upstream from the exit gore, at which a driver begins to diverge from the freeway.

DG - Deceleration in Gear Zone in which the vehicle decelerates prior to braking.

DB - Deceleration While Braking Zone in which braking occurs in order to reach a reduced speed dictated by the geometrics, terminus, or traffic conditions on the off-ramp.

FIGURE 5 The exit process (*2*).

factors. The models segmented the elements of the speed-change maneuver into additional components. Figure 5 is a diagram of the exit process. Table 2 gives the design values for the deceleration lane length (DG + DB in Figure 5) recommended in the NCHRP report.

The NCHRP report recommends deceleration lengths that are significantly higher than AASHTO.

Entrance Ramps

Figure 6 shows some of the design elements associated with entrance ramps. The most critical design is the distance provided to accelerate from the design speed of the ramp again, as with the exit ramps, dependent on the ramp radius and the rate of ramp superelevation—and the design and average running speed of the highway. A vehicle should be able to accelerate at least to the average running speed of the highway before the driver begins the merging maneuver. Table 3, reproduced from the Green Book (*1*), indicates current AASHTO minimum acceleration lengths for entrance terminals with grades of 2 percent or less. This table has also not changed from the values derived for the 1965 Blue Book (*6*).

As with deceleration lanes, the lengths are from the ramp curve to a point that is the end of the parallel lane or when the tapered ramp width decreases to 12 ft. The taper section for tapered ramps should remain unchanged with respect to the freeway pavement edge, and if the ramp type is a parallel design, a 300-ft minimum taper length should be used for parallel ramps.

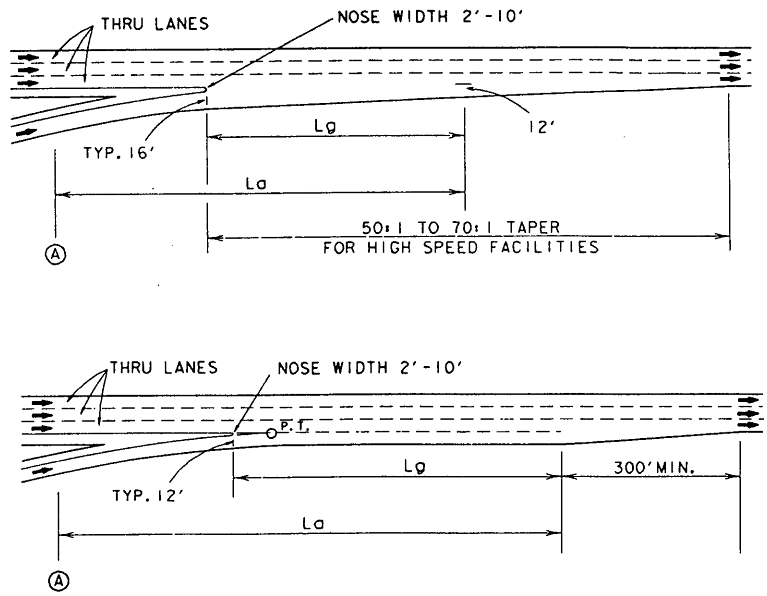
Although not a direct part of the speed-change lane, the ramp curve being used to enter the entrance ramp should be available to permit drivers to begin evaluating gaps in freeway traffic and to begin accelerating. If the ramp has a radius in the range of 800 to 1,000 ft, the motorist on the ramp or freeway will have an unobstructed view of freeway or ramp traffic, respectively.

Several states use the same design lengths for all deceleration lanes and all acceleration lanes, no matter what radius or design speed is used on the ramp curve entering or leaving the speed-change lanes. This consistency of design should aid driver familiarity.

TABLE 2 Deceleration DesignValues (*2*)

	Deceleration Lane Length (ft.)		
	V'_c (mph)		
V'_d (mph)	15	30	50
70	1,035	825	825
60	730	730	535
50	630	630	435

V'_d = Freeway diverge speed
V'_c = Ramp controlling speed



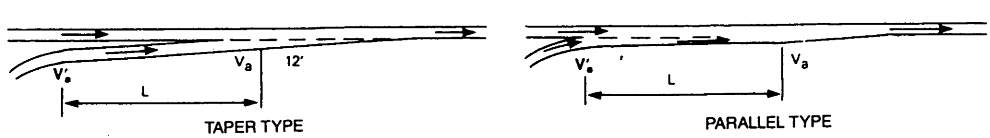
NOTES:

1. La IS THE REQUIRED ACCELERATION LENGTH AS SHOWN IN TABLE X-4 OR X-5.
2. POINT Ⓐ CONTROLS SAFE SPEED ON THE RAMP. La SHOULD NOT START BACK ON THE CURVATURE OF THE RAMP UNLESS THE RADIUS EQUALS 1000' OR MORE.
3. Lg IS REQUIRED GAP ACCEPTANCE LENGTH. Lg SHOULD BE A MINIMUM OF 300' TO 500' DEPENDING ON THE NOSE WIDTH.
4. THE VALUE OF La OR Lg, WHICHEVER PRODUCES THE GREATEST DISTANCE DOWNSTREAM FROM WHERE THE NOSE WIDTH EQUALS TWO FEET, IS SUGGESTED FOR USE IN THE DESIGN OF THE RAMP ENTRANCE.

FIGURE 6 Typical single-lane entrance ramps (*1*). *Top*, tapered design; *bottom*, parallel design.

TABLE 3 Minimum Acceleration Lengths for Entrance Terminals with Flat Grades of 2 percent or Less

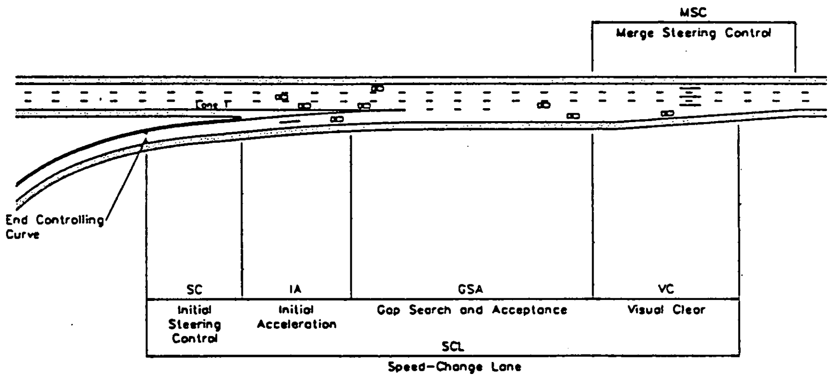
Highway		Acceleration Length, L (ft) For Entrance Curve Design Speed (mph)								
		Stop Condition	15	20	25	30	35	40	45	50
Design Speed (mph)	Speed Reached, V_a (mph)	and Initial Speed, V'_a (mph)								
		0	14	18	22	26	30	36	40	44
30	23	190	—	—	—	—	—	—	—	—
40	31	380	320	250	220	140	—	—	—	—
50	39	760	700	630	580	500	380	160	—	—
60	47	1,170	1,120	1,070	1,000	910	800	590	400	170
70	53	1,590	1,540	1,500	1,410	1,330	1,230	1,010	830	580



Note: Uniform 50:1 to 70:1 tapers are recommended where lengths of acceleration lanes exceed 1,300 feet.

The nose of the entrance ramp, or "merging end," is the point where the area between the ramp's left edge of pavement and the freeway's right edge of pavement, assuming a right-hand ramp, is paved. The nose width varies from 2 to 10 ft depending on local design standards. The ramp pavement width opposite the nose varies from 12 to 16 ft. The total offset of the ramp's right edge (18 to 22 ft) and the ramp's taper ratio will determine the total length of a taper-type acceleration lane.

The NCHRP 3-35 model also developed design values for entrance ramps. Figure 7 is a diagram of the entry process, and Table 4 gives recommended design values developed for the acceleration lane length (IA and GSA in Figure 7). This distance corresponds to "La" in Figure 6.



SC - Steering Control Zone which involves the steering and positioning of the vehicle along a path by steering from the controlling ramp curvature onto the speed-change lane.

IA - Initial Acceleration Zone in which the driver accelerates to reduce the speed differential between the ramp vehicle and the freeway vehicles to an acceptable level for completing the merge process.

GSA - Gap Search and Acceptance Zone during which the driver searches, evaluates, and accepts or rejects the available lags or gaps in the traffic stream. This zone is the key component of the entry model.

MSC - Merge Steering Control Zone during which the driver enters the freeway and positions the vehicle in Lane 1. This zone, however, is not considered a determinant of the speed-change lane length.

VC - Visual Clear Zone which provides a buffer between the driver and the end of the acceleration lane. Once a driver reaches this zone, he must take one of two actions, either merge onto the freeway in a forced maneuver, or abort the merge process and begin to decelerate at a reasonable rate.

FIGURE 7 The entry process (*2*).

TABLE 4 Acceleration Design Values (*2*)

	Acceleration Lane Length (ft.)					
	v^1_o = 15 mph			V^1_o = 30 MPH		
	v^1r_1 (mph)			v^1_{r1} (mph)		
v^1_f (mph)	40	50	60	40	50	60
70	2,550	2,025	2,400	2,475	1,975	2,425
60	1,750	2,025	*	1,675	1,975	*
50	1,700	*	*	1,625	*	*

* For this design condition, use next lowest value of v_r

V'_f = Freeway speed

v'_o = Initial ramp speed

v'_{r1} = Ramp speed at beginning of GSA zone

INTERPRETATION

Although basic criteria and thereby design standards used by governmental agencies have not changed in more than 30 years, most researchers who have investigated the operational aspects of speed-change lanes have found current design elements to be acceptable for today's driving conditions.

A survey of nationwide design agencies indicates that two-thirds of the agencies use both taper- and parallel-type speed-change lanes, depending on location and freeway conditions. Nearly all agencies use deceleration lane lengths that equal or exceed AASHTO recommendations. The greatest design difference lies in acceleration lane lengths, which in some cases are less than AASHTO recommendations.

Most researchers indicate very few operational problems with deceleration lanes. However, the gore of exit ramps ranks high in the location of freeway accidents. Researchers also indicate some problems with driver gap acceptance occurring on entrance ramps. Both conditions have been attributed to the assumption that drivers do not know how to properly use, or just do not properly use, speed-change lanes.

Little literature was found dealing with the impact of traffic control devices, including signals, signing, and striping at freeway/ramp merge or diverge areas.

No research was found describing the effect of ramp metering on acceleration lane length and operation or the operation of speed-change lanes at night with or without roadway lighting. Additional research dealing with these types of ramp operation should be beneficial. Additional research would also be useful in evaluating the operational differences between urban and rural operation, right and left side ramps, and single and two-lane ramps.

REFERENCES

1. *A Policy on Geometric Design of Highways and Streets*. American Association of State Highway Transportation Officials, Washington, D.C., 1990.
2. *Speed-Change Lanes*. National Cooperative Highway Research Program Report 3-35. TRB, National Research Council, Washington, D.C., 1989.
3. *Vehicle Operating Characteristics on Outer Loop Deceleration Lanes of Interchanges*. Ontario Joint Highway Research Program Report 43. University of Toronto, Toronto, Canada, 1968.
4. A. Polus and M. Livneh. Comments on Flow Characteristics on Acceleration Lanes. *Transportation Research*, Vol. 21A, No. 1, 1987, pp. 39–46.
5. R. A. Lundy. The Effect of Ramp Type and Geometry on Accidents. In *Highway Research Record 163*, HRB, National Research Council, Washington, D.C., 1967.
6. *A Policy on Geometric Design of Rural Highways*. American Association of State Highway Officials, Washington, D.C., 1965.

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Two-Lane Loop Ramps: Operation and Design Considerations

Ross J. Walker

The literature on the operational, safety, and capacity aspects of existing two-lane loop ramps is reviewed, and an overview is provided. Using this information, design parameters are suggested until further research is available. Little research information is available on the operation and design of two-lane loop ramps. AASHTO covers single-lane loop ramps very well but does not mention two-lane loop ramps. A TRB literature search indicates that only Georgia has carried out research on two-lane loop ramps. The findings of the Georgia report are discussed along with observations of other two-lane loops in Toronto, Vancouver, and Florida. Generally, directional or semidirectional ramps are preferred for high-volume ramps. They provide more direct travel, easier operation, and higher capacity than loop ramps. However, where there is insufficient space, the two-lane loop ramp is a reasonable compromise. The observations show that two-lane loops operate well where they have good geometric properties. Entrance and exit transitions and ramp widths and curvature all have to be integrated to ensure safe and convenient operation.

Little research information on the operation and design of two-lane loop ramps is available. AASHTO covers single-lane loop ramps very well but does not mention two-lane loop ramps. A TRB literature search indicates that only Georgia has carried out research on two-lane loop ramps.

It appears that the first two-lane loop ramp was constructed on Highway 401 at Weston Road in Toronto in approximately 1966. This ramp has been in operation for 25 years. The high volumes that were first predicted have not materialized. It presently carries 1,100 vehicles in the peak hour, 10 percent of which are trucks.

EXISTING TWO-LANE LOOP RAMPS

Table 1 gives the dimensions of five two-lane loop ramps in operation today. Included are the pavement and shoulder width, radius of the controlling curve, and peak-hour volumes. These ramps are completely two lanes, including the exit and entrance terminals.

There are also many partial two-lane loops, where ramp metering and HOV and priority lanes are used. In these cases, the ramp proper becomes two lanes after a single-lane exit and continues as two lanes up to the ramp entrance where it tapers back to one lane. Table 1 indicates that there is little uniformity in the geometric dimensions of these ramps.

Delcan Corporation, National Engineering Technology Corporation, 16700 Valley View Avenue, Suite 260, La Mirada, Calif. 90638.

Highway 401 and Weston Road—Toronto

Figure 1 shows the configuration of the ramp that is part of a Parclo A interchange. It has spirals at each end leading into 20-degree curves, which are then compounded to a 38-degree controlling curve.

The pavement width is 24 ft with a 1.5-ft left shoulder and an 8-ft right shoulder. The peak volume is 1,100 vehicles per hour (vph). Pictures of this ramp are shown in Figures 2 through 4. Both lanes of the ramp are used, even at low volumes.

Figure 2 shows the two-lane exit on Weston Road where both lanes are dropped. There are signals immediately upstream from this picture, and the two lanes line up at the signalized intersection and continue through to the exit. The two-lane loop was designed to allow the exit to carry the 1,500+ volume forecast.

Figure 3 shows that both lanes of the ramp are used, and in this case the spacing between the vehicles seems to be adequate. However, as traffic continues around the loop, a W-beam guardrail is introduced 3 ft from the left edge of the pavement, just outside of view of Figure 2. This rail tends to restrict larger vehicles so that they crowd the right lane. The 24-ft pavement width is too narrow, and wider lanes would give better and safer operation.

Figure 4 shows the two-lane loop ramp entrance with the inside lane being carried on as the added lane to the freeway.

Highways 99 and 17—Vancouver, British Columbia

This ramp has a left shoulder width of 6 ft, a pavement width of 27 ft, and a right shoulder width of 6 ft. The controlling radius is 230 ft. The ramp has a uniform shape similar to Weston Road. Highway 17 has signals just east of the ramp exit, and therefore it operates as a Parclo A. The second lane is optional for ramp and through traffic on Highway 17. However, the optional lane is filled with ramp traffic during the peak hour. Highway 99 is a freeway.

This ramp operates well, as shown in Figures 5 through 8. The traffic is well spaced across the width of the ramp. During peak hour this ramp is subject to surge flows. Under these conditions the ramp carries 2,530 vph (15-min flows) and operates at 15 to 20 mph.

Florida Turnpike 821—Kendall Drive, Miami

This two-lane configuration has a good speed transition zone with 400-ft spirals and a controlling radius of 240 ft. It operates

TABLE 1 Geometry of Existing Two-Lane Loop Ramps

Location	Left Shoulder	Pavement Width	Right Shoulder	Radius	Peak Hour Volumes
	feet				vehicles p/h
Highway No. 401, Weston Road, Toronto	1.5	24.0	8.0	154	1,100
Highway 99, Highway 17, Vancouver, B.C.	6.0	27.0	6.0	230	2,530
Florida Turnpike 821, Kendall Drive, Miami	2.0	28.0	8.0	240	1,220
I-75N to I-285W, Atlanta	4.0	37.0	7.0	200	1,040
G 400 Holcomb Bridge, Atlanta	10.6	23.5	4.3	150	2,070

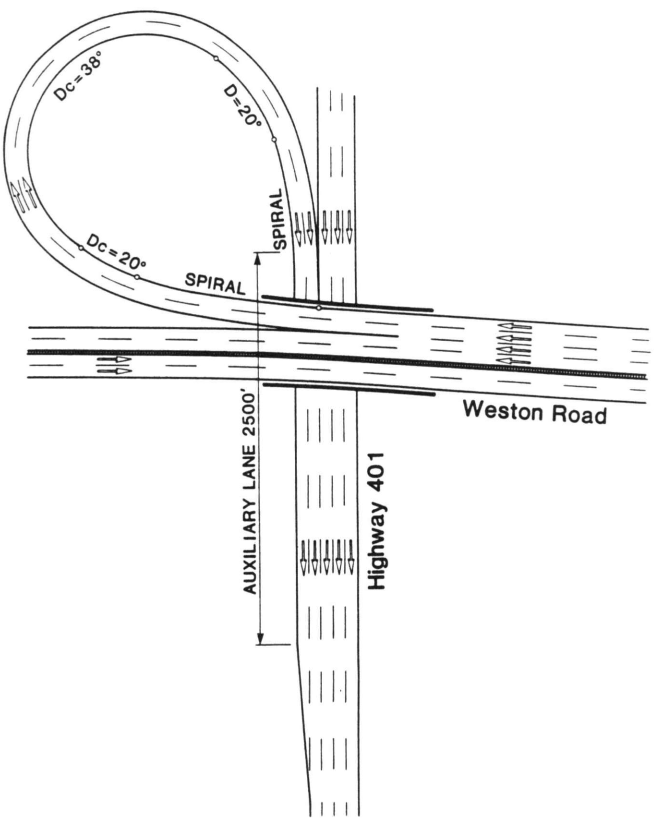


FIGURE 1 Highway 401 and Weston Road ramp.

FIGURE 2 Two arterial lanes approach exit to loop ramp (Toronto).

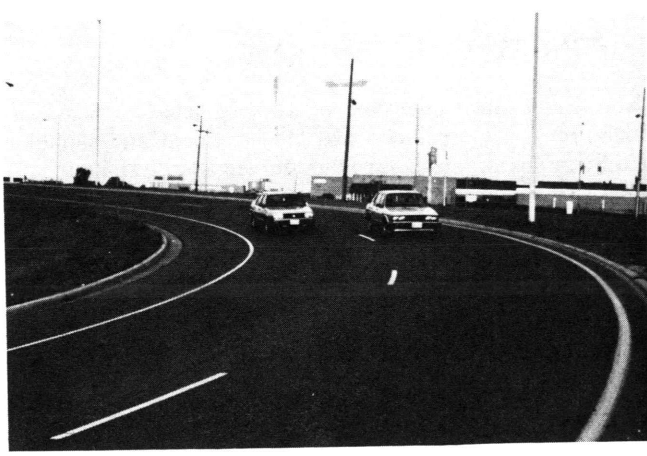


FIGURE 3 Spacing of vehicles on two-lane loop under low-volume conditions (Toronto).

as a Parclo B with a single exit off Kendall Drive before the structure as shown in Figure 9. The pavement width is 28 ft, the right shoulder width is 8 ft, and the left shoulder width is 2 ft. Present off-peak volumes are low because the ramp has only recently been opened to traffic. Even in low-volume periods, both lanes are being used.

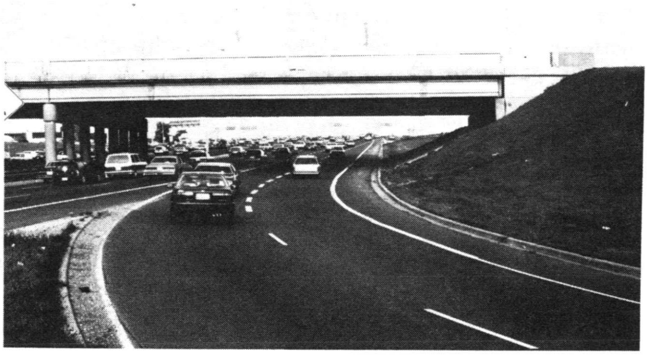


FIGURE 4 Two-lane entrance to freeway from loop ramp (Toronto).

FIGURE 5 Traffic exiting highway 17 to two-lane loop ramp (Vancouver).

FIGURE 6 Vehicle spacing on two-lane loop ramp (Vancouver).

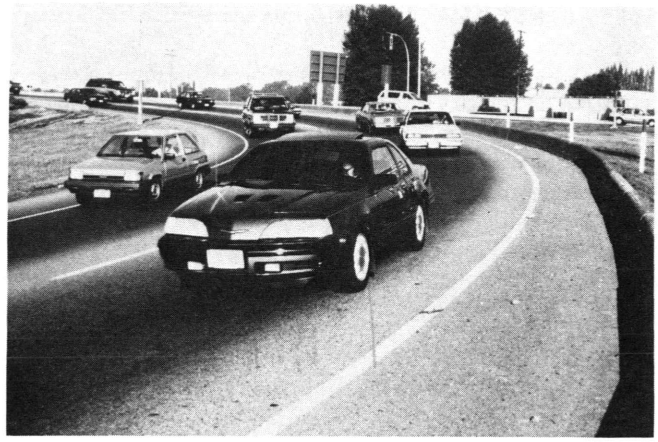


FIGURE 7 Vehicle position adjacent to trucks (Vancouver).

FIGURE 8 Vehicle spacing just before entrance (Vancouver).

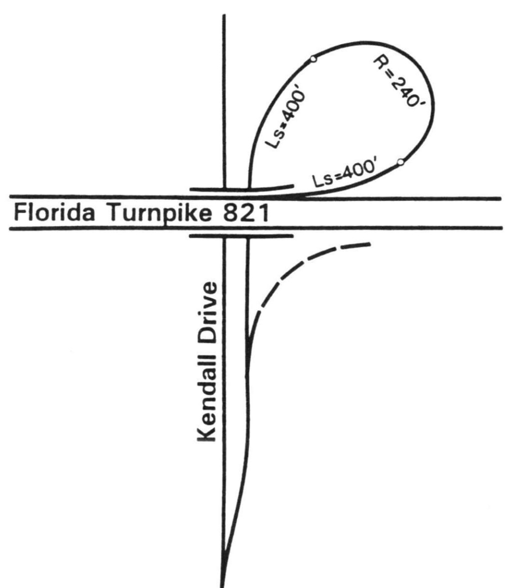


FIGURE 9 Florida Turnpike 821–Kendall Drive (Miami).

I-75N to I-285W—Atlanta, Georgia

Figure 10 shows the layout of the two-lane ramp after widening. The controlling curve has a radius of 200 ft with a left shoulder of 4 ft, a right shoulder of 7 ft, and a pavement width of 37 ft. The peak hour volume is 1,040.

Backups frequently occurred before the addition of a second lane. After construction, the two-lane loop ramp operated smoothly, with both lanes being used. The right lane carried more traffic than the left. Traffic was not heavy enough to really test the capacity of the two-lane loop. However, even with the 18 percent truck traffic, cars were able to pass and experienced no delays. The ample width of the ramp allowed the traffic to run smoothly.

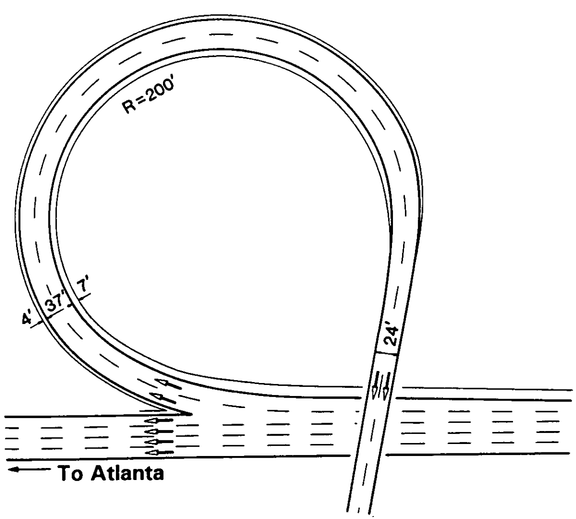


FIGURE 10 Two-lane loop ramp I-75 to I-285W.

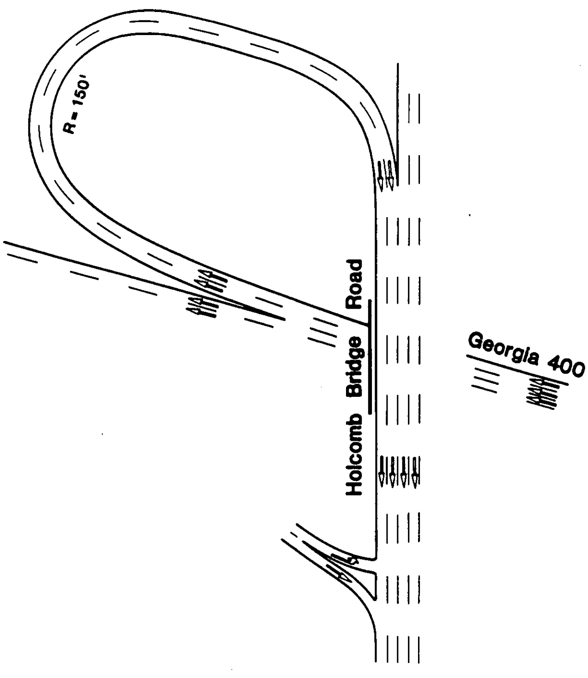


FIGURE 11 G 400 and Holcomb Bridge ramp.

G 400 and Holcomb Bridge—Atlanta, Georgia

The G 400 Holcomb Bridge ramp has an irregular shape as shown in Figure 11. It has a wide left shoulder of 10.6 ft, a modest right shoulder of 4.3 ft, and a narrow pavement width of only 23.5 ft. The peak-hour traffic is high at 2,070. Studies at this location indicated that the pavement width is too narrow and should be widened. Backups into the curve of the ramp were reduced. However, there are several encroachments from lane to lane, some due to vehicles traveling too fast. There were also incidents of following too closely and conflict between lanes in the curve.

ACCIDENT EXPERIENCE

The Highway 401–Weston Road ramp has only had six accidents in 3 years (1988 through 1990): one rear-end, two sideswipes, and three single-vehicle. Four accidents were in daylight and two at night. Five were in dry conditions and one was in wet conditions.

Before-and-after studies of the I-75N to I-285W and G 400 and Holcomb Bridge ramps have been carried out by Selph and Caylor (*1*). The before findings showed that the rear-end collisions at the I-75 to I-285 ramp occurred in the first two-thirds of the ramp. These could have been caused by two factors: (*a*) cars having to reduce speed abruptly because of trucks climbing the 3 percent upgrade of the ramp and (*b*) the lack of deceleration or transition distance between the nose and the controlling curve. Vehicles exiting from the high-speed I-75N have insufficient distance to decelerate to the ramp speed.

The before studies showed that rear-end collisions occurred on the G 400 ramp near the entrance to Holcomb Road. The longer speed transition of the exit allowed traffic to adjust to the ramp curvature. However, the problems at the entrance probably occurred because of the cyclical stop-and-go traffic due to the signal on Holcomb.

After-accident data also indicated an increase in angle-intersecting accidents on the exit section of the G 400 ramp. This ramp did not have an advisory speed sign, and the narrow width of the two-lane ramp left very little room for vehicles to maneuver. The Parclo B type loop with a direct exit from the freeway also required the high-speed traffic from the freeway to adjust very quickly to the slow ramp speed. These could be factors in the angle-intersecting accidents on this ramp. Further studies and research are needed to verify these assumptions.

DESIGN CONSIDERATIONS

Design features of a two-lane loop ramp can be broken into three main categories: the exit, the ramp, and the entrance. The following discussion suggests some design criteria for two-lane loop ramps on the basis of the author's observations and experience. Research is required to assess these criteria.

Exit Design

Exit From Arterial (Parclo A)

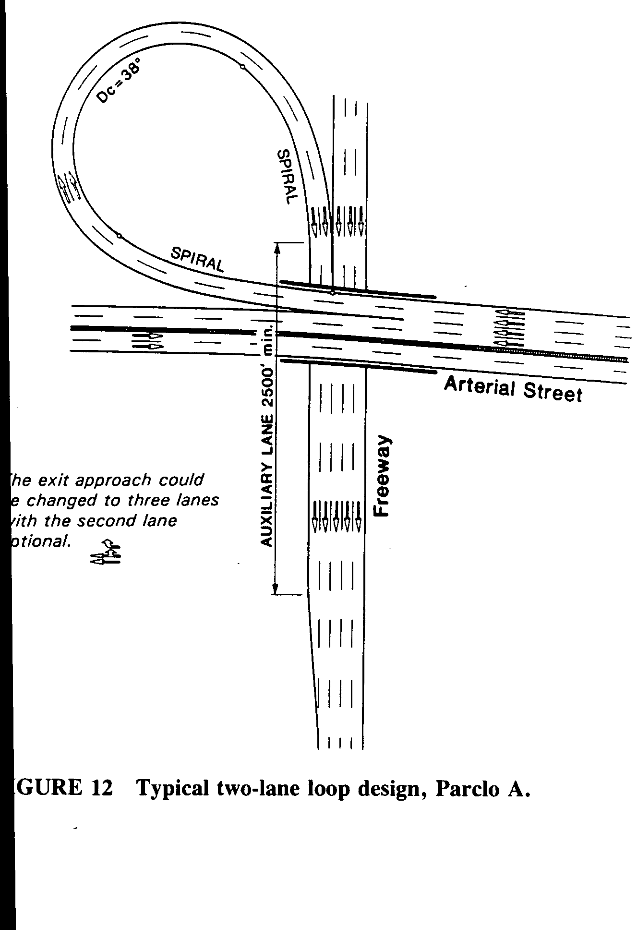
Figure 12 shows a typical Parclo A two-lane loop ramp exiting from an arterial street. Two lanes are dropped at the exit to increase the capacity of the ramp. The two lanes continue back through the signalized intersection. This allows traffic to line up in the proper lanes at the intersection and increase the flow to the ramp. The exit approach could be changed to a three-lane approach with the second lane optional. This would provide better lane balance but would reduce the capacity of the ramp. The selection of either of these designs would depend on the intersection spacing and capacity requirements.

Overhead lane signs are required at the signal to ensure that traffic is positioned in the proper lanes. Must Exit signs would also be required.

Exit From a Freeway (Parclo B)

A single exit before the structure is recommended as shown on Figure 13. The two-lanes exit requires an auxiliary lane upstream for 2,500 ft to develop exit capacity. After the exit, the ramp splits two to one with two continuing to the loop ramp.

A 150-ft radius (25 mph) is normally used for restricted urban conditions. A 230-ft radius (30 mph) could be used in more open or rural situations to reduce the speed transition by 5 mph.



GURE 12 Typical two-lane loop design, Parclo A.

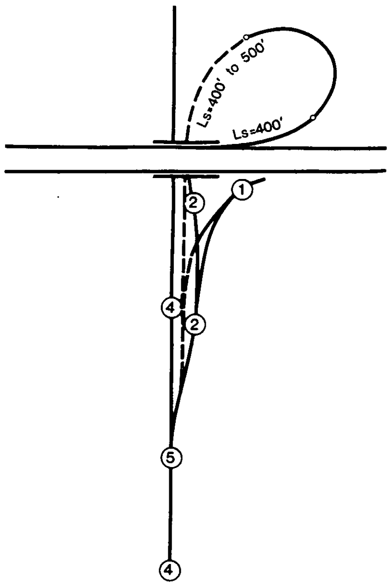


FIGURE 13 Typical two-lane loop design, Parclo B.

Traffic exiting from the freeway will be traveling in the 60- to 70-mph range and will have to decelerate to 25 to 30 mph at the loop ramp. If the configuration is as shown on the dashed lines of Figure 13, there will be a tendency for drivers to speed up instead of slowing down. They will then have difficulty negotiating the sharp curvature of the loop.

When the two lanes of traffic have to negotiate the sharp curvature of the loop, the design has to provide for as smooth an operation as possible. Any erratic maneuvers are much more hazardous with the two lanes. This means that the geometric design has to provide an alignment that allows drivers to transition to the very slow speed of the ramp. If they enter the loop traveling too fast, they will cause accidents.

The curvilinear design (solid line) helps alleviate this problem because drivers will adjust their speed gradually over the whole ramp. A long spiral is preferred at the entrance to the loop ramp to assist in the speed transition. Drivers recognize the curvature of the spiral and can adjust their speed accordingly.

Ramp Design

Figures 12 and 13 show a desirable ramp layout with a spiral transition from the exit to the controlling curve of the ramp. This curve is followed by a spiral transition to the entrance area.

Table 2 gives recommended pavement and shoulder widths for a two-lane loop ramp for 25 mph (150-ft radius) and 30 mph (230-ft radius) ramp design speeds. The pavement width is important to provide good lateral clearance between vehicles and to allow for smooth operation. The shoulder widths, 8-ft right shoulder and 4-ft left shoulder, allow ample space so that drivers do not feel crowded while making the tight radius turn. These dimensions will allow traffic to flow smoothly and provide the required capacity.

TABLE 2 Two-Lane Loop Ramps—Recommended Pavement Widths

Traffic Condition[a]	Left Shoulder	Pavement Width	Right Shoulder	Radius
	feet			
A	4	26	8	150 - 230
B	4	28	8	150 - 230
C	4	30	8	150 - 230

[a] Traffic Conditions A, B, C - See Table X-3 AASHTO - A Policy on Geometric Design of Highways and Streets *(2)*.

Superelevation will depend on the location. However, a 0.08 ft/ft maximum would be considered in snow conditions. Vehicles tend to overdrive ramps, and the 0.08 ft/ft will assist in allowing for this. Normally Parclo A ramps are in a downgrade. Where the ramp is on an appreciable upgrade and there is any chance of trucks' speeds being reduced to a crawl, 0.06 ft/ft could be considered. This requirement would allow for side slippage of vehicles at slow speeds on ice. The compromise has to be established between the extra super, which is used all the time, and the number of times that ice may be a problem. Values higher than 0.08 ft/ft in warmer climates would be desirable.

Entrance Design

The standard two-lane entrance design from AASHTO should be used, which will allow for an auxiliary lane of 2,500 ft for turning volumes of 1,500 to 2,000 vph and 3,000 ft for volumes in excess of 2,000 vph.

Whereas a single-lane ramp can accommodate up to 1,500 vph, it is unlikely that a single exit or entrance can, unless a line is dropped at the exit or added at the entrance. A lane drop at the exit would not provide good lane balance, and therefore a two-lane exit is more desirable for volumes over 1,000 vph. A single lane at the entrance would be acceptable.

SUMMARY

Generally, directional or semi-directional ramps are preferred for high-volume, two-lane ramps. They operate at higher speeds and tend to be safer. However, where there is insufficient space or a need to increase the capacity of an existing one-lane loop ramp, the two-lane loop ramp is a reasonable compromise solution. It will provide the required capacity, although not as good service, as the directional ramp.

With proper exit, entrance, and speed transition zone design, a loop ramp can carry up to 2,000 vph in a safe and reasonable manner. The capacity will depend on the approach road configuration. The recommended widths in Table 2 should ensure smooth flow. Care should also be taken to allow adequate auxiliary lane lengths and acceleration lanes when entering the mainline on an upgrade. The 2,000 vph could require a change in the basic number of lanes or certainly addition of an auxiliary lane as well as care in establishing lane balance.

Volumes of up to 2,500 have been experienced as indicated in Table 1. However, this ramp operates at Level of Service F (15 mph).

REFERENCES

1. S. D. Selph and T. N. Caylor. *Evaluation of Two-Lane Loo Ramps*. Research and Development Bureau, Office of Materia and Research, Georgia Department of Transportation, Atlanta
2. *A Policy on Geometric Design of Highways and Streets*. America Association of State Highway and Transportation Officials, Wash ington, D.C., 1990.

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Geometric Design of Metered Entrance and High-Occupancy Vehicle Bypass Ramps

TIMOTHY J. LOMAX AND CHARLES A. FUHS

Freeway entrance ramp meters and high-occupancy vehicle bypass lanes are being used more frequently as one tool to address increasing traffic congestion. The geometric design standards and practices of states that operate these two technologies are reviewed. Signing, marking, signalization, enforcement area designs, and operating policies are also summarized to identify the significant aspects relating to the geometric design standards. Some of the installations are designed to less-than-desirable geometric standards as a result of constrained urban rights-of-way. Additional information or traffic control devices are often necessary to overcome these deficiencies.

Freeway entrance ramp meters are being used more frequently as one tool to address increasing traffic congestion. Allowing fewer vehicles to enter a freeway during peak traffic periods reduces the potential for accidents, encourages the use of streets for short distance trips, and can allow the freeway to move more vehicles at higher speeds. Regulating the flow of vehicles onto the freeway also eliminates the effect of upstream signalized intersections, which tend to produce platoons of vehicles separated by little or no volume. Entering vehicles can merge more easily with freeway traffic if the pressure from following vehicles is alleviated. A recent survey (*1*) of state departments of transportation (DOTs) indicates approximately 1,800 metered freeway entrance ramps in operation in North America.

Bypass lanes at metered entrance ramps represent a relatively low-cost priority treatment that can provide travel time savings to high-occupancy vehicles (HOVs). In some cases the amount of time saved on an HOV bypass of a metered ramp is sufficient to induce increased use of carpooling and transit, but more frequently it is a small incentive that can be combined with other measures to make high-occupancy travel more attractive. Approximately 450 HOV bypass lanes were in operation in 1991 (*1*).

This paper focuses on the current practice in design of these two freeway interchange elements. Most of the ramp meters and HOV bypass lanes that have been constructed to date are the result of "retrofit" design policies that made the most efficient use of space and funding. Where there are design standards or guidelines, it is frequently difficult to provide desirable design dimensions for freeway ramps that were not designed for the different operating characteristics of a metered entrance ramp. This paper, therefore, presents both "desirable" and "retrofit" design practice to illustrate both the application of typical design standards and what has been successfully operated by local transportation agencies. Whereas desirable standards are important to the design process, it is also important to note in a summary of the state of the practice the designs that appear to work well.

The American Association of State Highway and Transportation Officials guidelines (*2–4*) provide some guidance for the design of metered entrance ramps and HOV bypass lanes. Additional research may be required, however, to identify acceptable trade-offs between desirable standards, project benefits, and implementation concerns.

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ELEMENTS OF METERED RAMPS AND HOV BYPASS LANES

The general configuration of the design elements of a metered freeway entrance ramp with an HOV bypass lane is shown in Figure 1. The figure shows a ramp from a cross street. A paint stripe is used to separate the HOV and general traffic flows from a point upstream of the queue to downstream of the ramp meter. Both lanes are metered in Figure 1. The lower signal heads face the vehicles at the stop bar, and the upper signals reinforce the Signal Ahead When Flashing sign and alert approaching vehicles.

Whereas there is no single arrangement of these elements that is common to all ramp metering projects, Figure 1 provides an overview of the design, signing, and marking details that are discussed in this paper. The entrance ramp could connect to a frontage road with a separate ramp for HOVs (Figure 2), or there could be two lanes available for queue storage of general-purpose vehicles. Some projects have used a buffer island to separate the HOV traffic and provide a location to install the signals. Most operating projects do not meter the HOV bypass lane as it enters the freeway. The HOV lane could be on either the right or the left side of the general traffic ramp lane.

REVIEW OF STATE PRACTICE AND GUIDELINES

The states of Arizona (*5,6*), California (*7,8*), Michigan (conversation with Michigan DOT officials, November 1991), Oregon (*9*), Virginia (*10*), and Washington (*11*) have specific

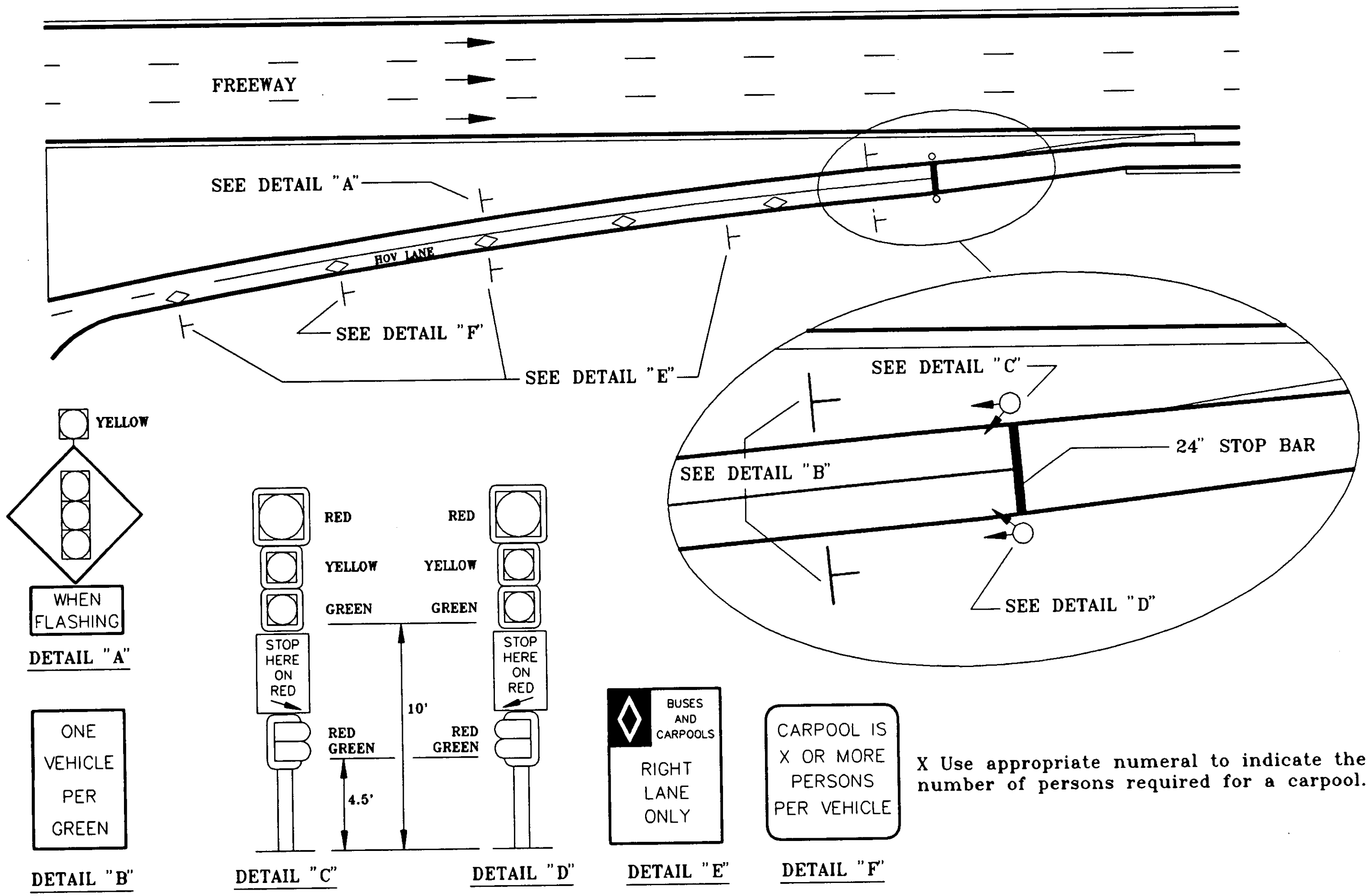

FIGURE 1 **Typical ramp meter and HOV bypass lane installation (*2*).**

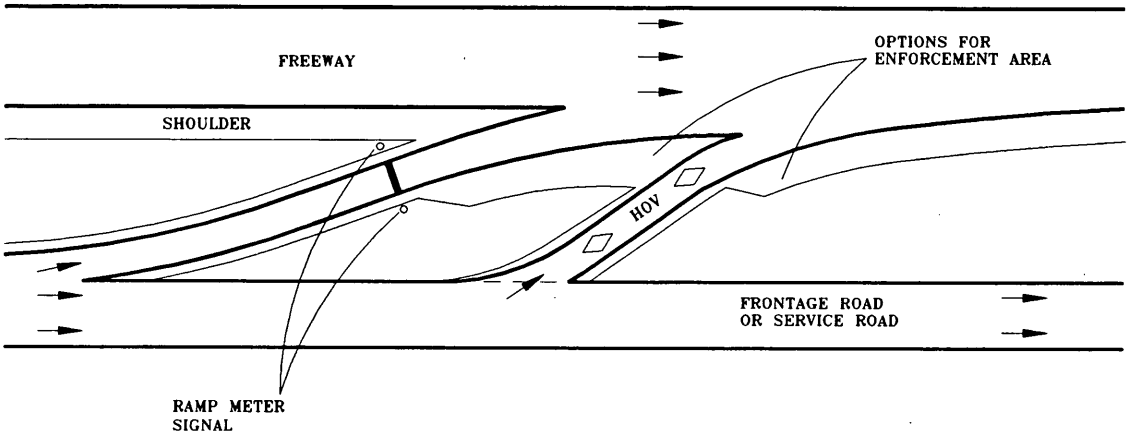

FIGURE 2 Separate HOV bypass lane at metered entrance ramp (*12*).

documents related to the design of ramp metering or HOV bypass lanes, or both. Several other states with operating ramp meters that were contacted for this paper had only general guidelines (conversations with officials of Colorado, Illinois, and Minnesota DOTs, November 1991) or used AASHTO documents as the source of design guidance. The major items specified in these documents included the following:

- Width of the entrance ramp lane and shoulders,
- Length of the entrance ramp and the merge area to the freeway lanes,
- Signalization,
- Signing and marking,
- Queue storage considerations,
- Separation of HOV and general ramp traffic, and
- Enforcement of HOV restrictions.

This section presents a summary of the design guidelines and existing practice of the state DOTs involved in ramp metering and HOV bypass lane operation.

Width of the Metered Entrance and HOV Bypass Ramps

Two methods of providing space for metered entrance ramps have been used on freeways in the United States. The more common method is for the metered lane or lanes to be striped for use during the day. If two lanes are provided to handle peak traffic demands, there is a significant amount of unused capacity during off-peak times.

An alternative method used in Minnesota (conversation with Minnesota DOT officials, November 1991) does not require new striping or additional lanes but rather uses signs and flashing lights to alert motorists to the need to form two lines on the ramp when metering is in operation. The signing directs the motorists to use the main ramp pavement and the paved shoulders to separate into two traffic queues during the low-speed operation during metering.

The desirable width reported most frequently for a one-lane ramp includes a 4-ft left shoulder, a 12-ft lane, and an 8-ft right shoulder. The range of desirable total ramp widths was between 20 and 27 ft (Table 1). These ramp widths are typical of those used in the initial freeway design; the introduction of ramp metering in these cases did not require any significant construction.

Two-lane metered ramp designs are usually installed in cases where a single-lane ramp would not provide sufficient storage capacity for the ramp queue that develops at the desired metering rate. If space permits, the design is similar to a regular two-lane ramp, with a 4-ft left shoulder, two 12-ft lanes, and an 8-ft right shoulder (which can be widened to three lanes by restriping). Some states use a ramp width of as little as 26 ft (two 13-ft lanes with no shoulders) to provide the additional storage capacity of two lanes.

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Priority treatments for high-occupancy vehicles at metered entrance ramps are provided as both separate ramps and additional lanes to existing general-purpose ramps. The separate ramp locations are designed with single-lane ramp design standards. The more common case of a lane being added to an existing ramp is usually handled using a paint stripe to separate HOV traffic (Figure 3). Most existing installations and design guides follow the dimensions reported for two-lane ramps, with one of the lanes being designated for HOVs. The Minnesota DOT (conversation with Minnesota DOT officials, November 1991) uses a 6- to 8-ft-wide buffer island (Figure 4) to separate two general ramp lanes from a 16-ft-wide HOV lane. The island is 300 to 500 ft long and is also used to provide a location to install the ramp meter signals for one general-purpose lane and for the HOV lane.

TABLE 1 Design Guidelines for Width of Metered Entrance Ramps

State	One-Lane Ramp	Two-Lane Ramp	Three-Lane Ramp
Arizona	22' Des.	24' Ret. 28' Des.	NS
California	24' (if less than 900 veh. per hour)	36' (if more than 900 veh. per hour)	40'
Colorado	18' Ret. 20' Des.	26' Ret. 32' Des.	8' island[2] 22' HOV lane
Illinois	20'[1]	None	None
Michigan	NS[1]	None	None
Minnesota	None	18' absolute min 22' Ret. 26' Des.	6'-8' island[2] 16' HOV lane
Oregon	26'	26'[3]	40'[3]
Virginia	NS	34'	
Washington	27'	36' Ret. 38' Des.	48'

Des. - Desirable dimension
Ret. - Retrofit dimension
NS - Not specified
None - No ramps of this type are in operation

[1] Width is determined by existing freeway entrance ramp
[2] Dimensions added to two-lane entrance ramp for HOV bypass lane
[3] Dimensions are currently being reviewed and may be increased

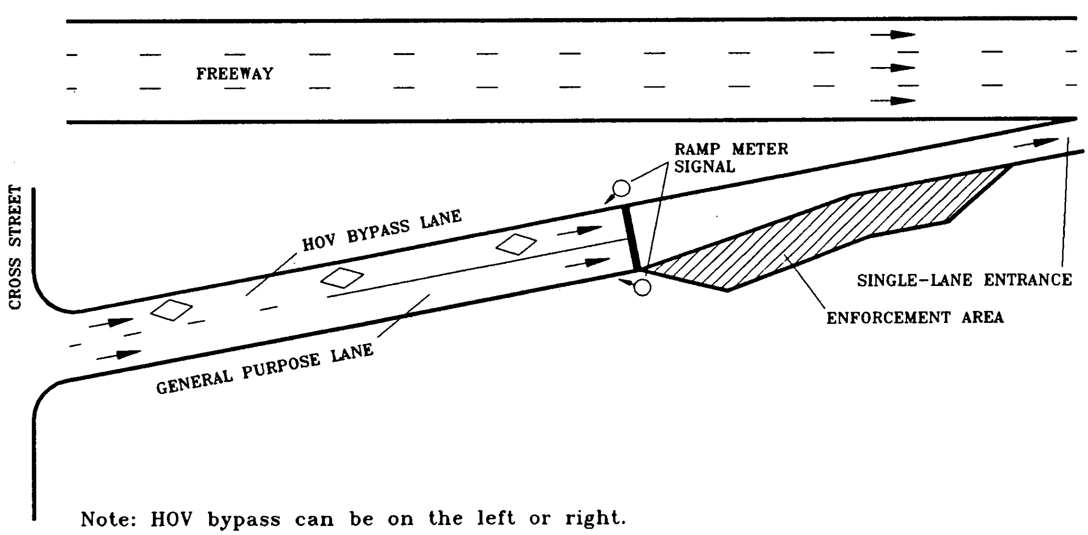

FIGURE 3 HOV bypass lane adjacent to general-purpose ramp lane (2).

Length of the Metered Entrance Ramps

The lengths of three design elements were specified in the design guides to illustrate metered entrance ramp design. Those elements were

- Cross street to the beginning of the ramp,
- Queue storage length on the entrance ramp, and
- Length from the stop bar to the freeway entrance gore point.

As with many aspects of metered entrance ramp designs, however, these three elements are heavily influenced by the general-purpose ramp design that was installed originally. The design process of metered ramps and HOV treatments in these cases usually focuses on dividing the available length to accomplish the most effective design. Factors considered in this design balancing process include cross street signal operation, available queue storage length, type of ramp meter operation (e.g., traffic responsive or fixed-time), enforcement area requirements associated with HOV restrictions, and accelera-

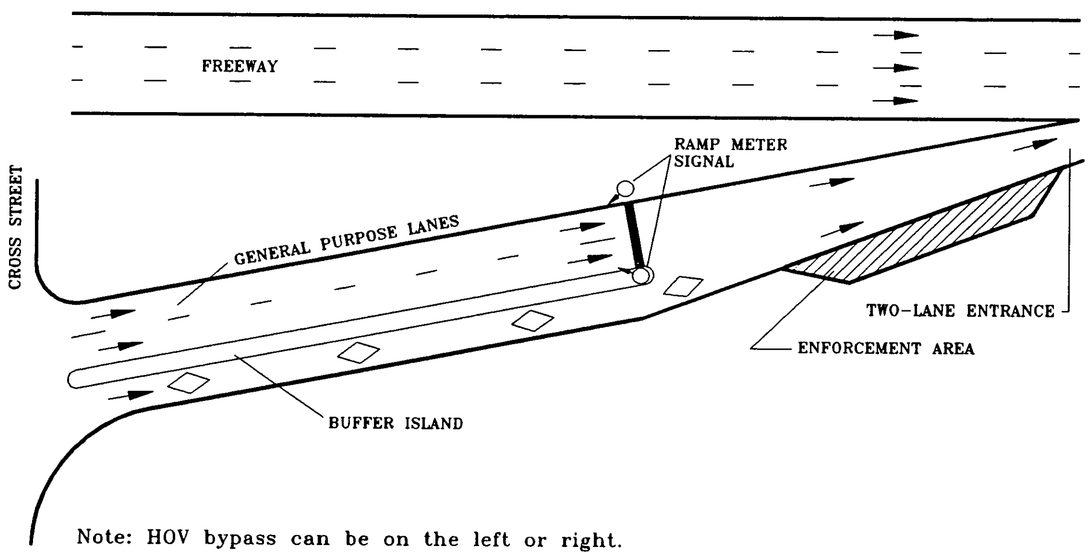

FIGURE 4 HOV bypass lane separated by buffer island from general-purpose ramp lane (*2*).

tion distance required to attain safe merging speed with the freeway traffic.

The distance from the cross street to the beginning of the entrance ramp is a dimension associated with freeway entrance ramps that begin from a frontage road or side street, rather than connecting directly to the cross street. The Minnesota DOT (conversation with Minnesota DOT officials, November 1991) has the only guidelines (Table 2) that include this dimension and lists 150 ft as a minimum distance, with 300 ft being desirable. This distance provides additional storage capability if the vehicle demand exceeds the metering rate by more than the amount that can be handled on the entrance ramp.

The desirable case, however, is for the queue to be contained on the entrance ramp. Street operating condition and safety can be increased if the queue does not interfere with the street traffic. The dimensions for required queue storage will vary on the basis of the freeway capacity and vehicle

TABLE 2 Design Guidelines for Length of Metered Single-Lane Entrance Ramps

State	Section of Entrance Ramp		
	Cross Street to Ramp	Queue Storage	Stop Bar to Freeway Gore
Arizona[1]	NS	760' Ret. 1010' Des.	340' 40 mph design 450' 45 mph design
California	NS	1000' Des.[2]	660' (30:1 taper)[3]
Colorado	NS	NS[4]	NS[5]
Illinois	NS	NS	300'
Michigan	NS	NS	250'[6]
Minnesota	150' Ret. 300' Des.	300' Ret. 500' Des.	250'-300' Ret. 400 Des.
(if HOV lane)			500'-600' (15:1 taper)
Oregon	NS	NS	250'
Virginia	NS	400' Ret. 1400' Des.	300'-350'
Washington	NS	500' Ret. 1000' Des.	700' Des. (50:1 taper)

Des. - Desirable dimension
Ret. - Retrofit dimension
NS - Not Specified; usually determined by site investigation and existing ramp design
[1] Document notes a desirable total distance from the center line of the cross street to the freeway gore of 1400 feet
[2] For three-lane entrance ramps and ramps with peak-hour volume exceeding 1500 vehicles, also need 1000' acceleration lane
[3] Document recommends study of individual ramp volumes to determine storage needs; allows use of cross street lanes to provide for adequate storage
[4] Provide maximum length possible for each site
[5] Put stop bar as close to freeway gore as possible and extend acceleration lane on freeway as needed
[6] Metered ramps are on a depressed freeway (ramps are on a downgrade approach)

demand and, therefore, are very localized design decisions. The retrofit distance referred to in the design guidelines ranges from 300 to 750 ft from the stop bar to the access street. The maximum or desirable distance ranged from 500 to 1,400 ft.

The type of ramp metering operation also affects the amount of queue storage required. Where traffic-responsive ramp metering is used, a higher-than-normal level of congestion on the freeway mainlanes could result in a longer queue than can be handled on the ramp. A queue detector induction loop located near the beginning of the entrance ramp can be used to modify the metering rate so that the queue does not impede operation on the cross street or frontage road.

The distance provided from the stop bar to the freeway lanes is a function of the speed that an entering vehicle needs to achieve at the gore point and the design of the ramp. The design guidelines identified in the preparation of this paper included a requirement that the speed at the end of the ramp be within 5 to 15 mph of the freeway speed. The distance that results from this requirement depends on the grade of the entrance ramp approaching the freeway. An additional consideration used by many DOTs is that the distance along the ramp may be reduced if there is an auxiliary lane present downstream of the entrance ramp junction. The auxiliary lane is used in these cases as an extended acceleration lane.

The reduced dimensions used by state DOTs for the location of the stop bar are those implemented with a downstream auxiliary lane; those dimensions ranged between 250 and 350 ft. The desirable distance for ramps without HOV bypass lanes ranged from 450 to 700 ft from the stop bar to the freeway gore.

Multilane ramps and ramps with HOV bypass lanes require more distance to merge the two or more lanes before they join the freeway mainlanes. The Minnesota practice (conversation with Minnesota DOT officials, November 1991) allows for at least a 15:1 taper (and desirably 45:1) to reduce the ramp to one entering lane. At HOV bypass locations where buffers are applied to segregate the parallel traffic streams, the minimum condition translates to a transitional distance of 500 to 600 ft. California's ramp meter guidelines (*7,8*) call for a 30:1 taper, with an additional 1,000 ft acceleration lane (12 ft wide) required beyond the ramp/freeway gore for three-lane entrance ramps (e.g., two general lanes and one HOV lane) or for ramps with high traffic volume (more than 1,500 vehicles in the peak hour). The Washington DOT guidelines (*11*) list a 50:1 taper beginning at the stop bar and continuing until the outside edge of the ramp pavement is within 2 ft of the auxiliary lane or freeway lane as the method of channelizing traffic flows from entrance ramps to freeway mainlanes. Some states, Colorado among them (conversation with Colorado DOT officials, November 1991), use a red indication extension on the general ramp lanes to provide an easier merge for HOVs.

As additional guidance to ramp designers, the Arizona DOT *Design Procedures Manual* (*5*) indicates that the desirable distance from the centerline of the cross street to the freeway is 1,400 ft.

Signal Design Considerations

Signal design on metered ramps should consider four functions of the ramp meter signalization:

- Signal operating condition: A three-headed signal mounted high on a pole is used in many states to alert approaching motorists to the status of the ramp metering operation. The signal is operated in a green indication for some period before metering begins, is then switched to amber for a short period, and is then switched to red. The signal then alternates between red and green for normal operation.
- Metering traffic flow: A two-headed signal mounted low and next to the stop bar is used as the driver's signal in most cases of single-lane metered ramps. For multilane ramps the signal is mounted above and in front of the stopped vehicle or on a pole adjacent the vehicle.
- Enforcement: A signal head or red indicator facing down the ramp can be used to alert enforcement officers to the signal indication. The officers can enforce both the signal and occupancy violations from a point downstream of the signal with this "tattletale" signal indication.
- HOV lane metering: Most HOV ramp bypass lanes are not metered because of a concern about a lack of perceived times savings if buses and carpools are required to stop at a signal. The design guidelines in California (*8*) and Minnesota (conversation with Minnesota DOT officials, November 1991) refer to the use of a ramp meter on the HOV bypass lane to reduce the speed differential between priority and nonpriority ramp traffic. Oregon also applies ramp metering to HOV bypasses. The meter is set at a relatively rapid rate so that HOV traffic is not significantly delayed. The meter has also been credited with increasing public acceptance of the bypass lane by requiring all vehicles to stop for at least some time and by reducing the presence of high-speed traffic immediately adjacent to traffic departing the signal at low speed.

Signing and Marking Considerations

The *Manual on Uniform Traffic Control Devices* (MUTCD) (*13*) includes a few examples of HOV lane signing practice. Many of the most frequently used signs at ramp meter installations, however, are not included in the MUTCD. A brief summary of the signs and their installation location is included in this design review to note the importance of the operational aspects of a successful ramp metering project. Additional guidance on ramp meter and HOV ramp signing in the MUTCD could improve the usefulness of that document for these facilities.

A sign located in advance of the entrance ramp alerts motorists to the presence of the ramp metering installation (Section 2C-17, MUTCD). This sign carries the diagram of a signal or the message Ramp Metered or, if implemented with a flashing signal, Ramp Metered When Flashing, Prepare To Stop When Flashing, or Signal Ahead When Flashing. The signing guidelines suggest that this sign should be located so that motorists can understand the message and stop before reaching the end of the ramp meter queue, especially in situations with limited sight distance to the end of the queue.

The One Vehicle per Green sign is used to notify motorists approaching the ramp meter and is located near the stop bar. In some states, when insufficient queuing space exists and more capacity is needed at the ramp meter, two vehicles will be allowed to proceed on each green. The additional message Each Lane is used where appropriate.

Stop Here on Red (Section 2B-37, MUTCD) is used with an arrow pointing to the stop bar to reinforce the meaning of the ramp meter signal. Wait Here for Green is also used to convey the same message in Illinois.

A special sign used in Minnesota (conversation with Minnesota DOT officials, November 1991) alerts motorists to the change necessary for peak-period operation. The Minnesota DOT has designed the metered ramps to operate as one lane during the off-peak periods and as a two-lane ramp during the metering operation. The sign carries the message Form Two Lanes When Metered and is located at the beginning of the ramp.

A lane reduction transition sign (Section 2C-19, MUTCD) and an optional Lane Ends sign are used downstream of the ramp meter signals on multilane installations to warn motorists of the reduction in the number of ramp lanes. The number of lanes is reduced to one before the ramp traffic merges with the freeway traffic.

HOV bypass lanes require signing to alert motorists to the restricted nature of the facility. Signs such as Right/Left Lane Carpools 2 or More Only When Metered (with some time period noted) or Carpools 2 or More Only (for 24-hr use) (Section 2B-20, MUTCD) are used with the diamond symbol to identify the restrictions.

The striping necessary to implement a ramp metering project is not a significant change from the usual ramp striping requirements. A stop bar is usually installed at the ramp meter signal. If a one-lane ramp meter project is placed on a full-width entrance ramp, chevron striping or jiggle bar tiles may be used to channelize the traffic flow and discourage motorists from passing the queue on the ramp shoulder. The diamond symbol is used on the HOV bypass lane pavement to reinforce the regulatory signing.

In addition to the signing that is used to designate ramp meter bypass lanes for HOVs, Car Pool Only markings have been installed on the HOV lane. A solid 8-in.-wide paint stripe has also been used in many states to separate the general traffic flow from the HOVs where there is no buffer or island provided to discourage vehicles from crossing into the HOV lane. On many operating projects, however, there has been no special striping involved in the HOV lane implementation other than that required to restripe a one-lane ramp into a two-lane ramp.

Enforcement of Ramp Meter and HOV Bypass Lanes

Several design elements and operating policies associated with metered ramps and HOV bypass lanes can assist the enforcement of the metering and HOV restrictions. The treatments used for enforcement of both are relatively similar and are discussed in the following section of this paper. Among the items considered by state DOTs and local police agencies are the following:

- Public information campaign: The successful public information programs have presented the goals and operating strategies of the ramp meter system and explained the traffic laws and penalties for violations. Among the items discussed are the function of the meters and the benefits of ramp metering implementation to the freeway system and to motorists. Briefings for traffic reporters have been used to convey the impact and benefits of ramp metering to individuals who have an effect on public opinion.
- Vehicle metering rate: The minimum acceptable metering rate varies by city and freeway condition, but the guidelines used in California are representative of the experience of operating agencies (*1,6,7*). The theoretical rate of vehicle flow through a meter is about 850 vehicles per hour per lane. The rate can be increased to 1,000 to 1,100 vehicles per hour per lane if two vehicles are allowed for each green. If the metering rate is faster, control over the ramp and the benefit to the freeway are diminished. Metering rates below 240 vehicles per hour (15-sec intervals between green signals) usually result in high violation rates and disrespect for the concept.
- Placement of enforcement areas: The success of a ramp metering project depends on the voluntary compliance of motorists. Early in the project, however, the presence of enforcement officers can significantly reinforce the meaning of the signs, signals, and markings. A paved enforcement area can encourage patrolling enforcement officials to provide this assistance to the concept. The involvement of enforcement officials at an early stage of the planning and design process can improve the usefulness of the enforcement areas and can increase the cooperation that is necessary for effective enforcement.
- Enforceable signs, signals, and markings: Traffic control devices that are consistent with the motor vehicle code and supported by the court system are important determinants for police support and, therefore, of the success of the project. California now posts the value of the fine at metered ramps; this has been effective in managing violations.

REVIEW OF AASHTO GEOMETRIC POLICY

The AASHTO *Policy on Geometric Design of Highways and Streets* (*4*) contains no specific guidelines for the implementation of either entrance ramp metering or HOV bypass ramps. AASHTO has produced two documents (*2,3*) that provide information concerning the design of HOV facilities and some information on ramp metering in general.

This section summarizes the recommendations of the more recent policy guidelines with regard to HOV bypass lanes, as well as the general remarks regarding metered entrance ramps.

Design of HOV Bypass Entrance Ramps

The 1992 AASHTO *Guide for the Design of High-Occupancy Vehicle Facilities* (*2*) discusses the need to balance the length along the entrance ramp for vehicle storage with the distance needed for vehicles to accelerate from the stop bar to freeway speeds. The 1990 AASHTO geometric design guide (*4*) uses a vehicle speed within 5 mph of the posted freeway speed at the end of an entrance ramp to design the acceleration lane. The 1992 HOV guide (*2*) suggests the use of the cross street for storage space if there is not sufficient length on the entrance ramp for both queueing and acceleration distances.

The AASHTO HOV guide (*2*) recommends that "the design of the ramp meter bypass . . . be determined by the conditions at each location." The HOV guide identifies a 12-ft lane with full ramp shoulders as the desirable cross section. The cross section should extend 300 ft beyond the metering signal to permit the priority and nonpriority traffic to merge before entering the freeway.

Bypass Lane Signing and Marking

The HOV guide (*2*) recommends the typical signing and markings found in the MUTCD (*13*) to identify the restrictions for HOVs. The regulatory signs summarized in the section on current state practice that are used to identify the appropriate lane and occupancy level are included in the guide. The use of the diamond symbol on the signs and on the pavement in the HOV lane is also recommended. A Yield sign may be necessary to assign the right of way downstream of a ramp meter location if both general and HOV traffic flows are present.

Bypass Lane Signals

The HOV guide (*2*) refers to both of the strategies used in current practice of signal design of HOV bypass lanes—metering and not metering the lane. The use of a meter on the lane, though decreasing the time advantage for buses and carpools, provides the opportunity to decrease the speed differential on the lane. The HOV guide recommends a Yield sign to assign right of way or a detector loop that will hold the general-purpose vehicles at the stop bar while the HOVs have a green indication, although this guideline has seldom been applied. No signalization is required for HOV bypasses that are on a separate ramp.

Enforcement Area Design

The dimensions associated with enforcement areas will vary according to the site characteristics, but the design in Figure 5 was included in the HOV guide to illustrate the principles involved. The California DOT (*7,8*) has a similar typical design in its HOV manual.

The area provides a refuge space for enforcement officers to observe driver obedience of the signal and HOV restrictions in a relatively low-speed situation. The area is also easily accessible by patrol officers on the freeway and could be connected to the street system (depending on local site conditions) to provide increased flexibility for patrol strategies. The HOV guide (*2*) mentions that violations increase when there is a clear view of the enforcement area and drivers can observe the presence or absence of patrol officers; some sort of screen may be desirable to reduce the need for continuous enforcement.

CONCLUSIONS

Freeway entrance ramp metering projects are in operation in approximately 1,800 locations in 13 states and one Canadian province (*1*). Whereas ramp metering is not a new concept [the first installation of ramp metering in the United States was in suburban Chicago in 1963 (*14*)], the factors required to make a project successful are present in only a limited number of cities and corridors. The ramp metering projects that have been implemented are on freeways that experience a significant amount of congestion, usually have a limited amount of right-of-way, and are sometimes installed as a stopgap measure because of an inability to widen the freeway.

As the fiscal and physical difficulties associated with widening freeways have increased, state departments of transportation have relied on transportation management techniques such as ramp metering to provide more vehicle movement capacity in an existing corridor. An addition to some projects that expands the role of ramp metering in increasing the person movement in a corridor has been a bypass lane for high-occupancy vehicles. This lane allows buses and carpools to achieve a time advantage relative to single-occupant vehicles and encourages the use of HOVs.

The implementation of intelligent vehicle/highway system technologies, which have as a basic goal the increased efficiency of the transportation system, may substantially increase the use of ramp meters and HOV bypass lanes to improve operation and provide for increased person movement capability.

One or both of these concepts, properly implemented and enforced, can improve the operation of a freeway corridor.

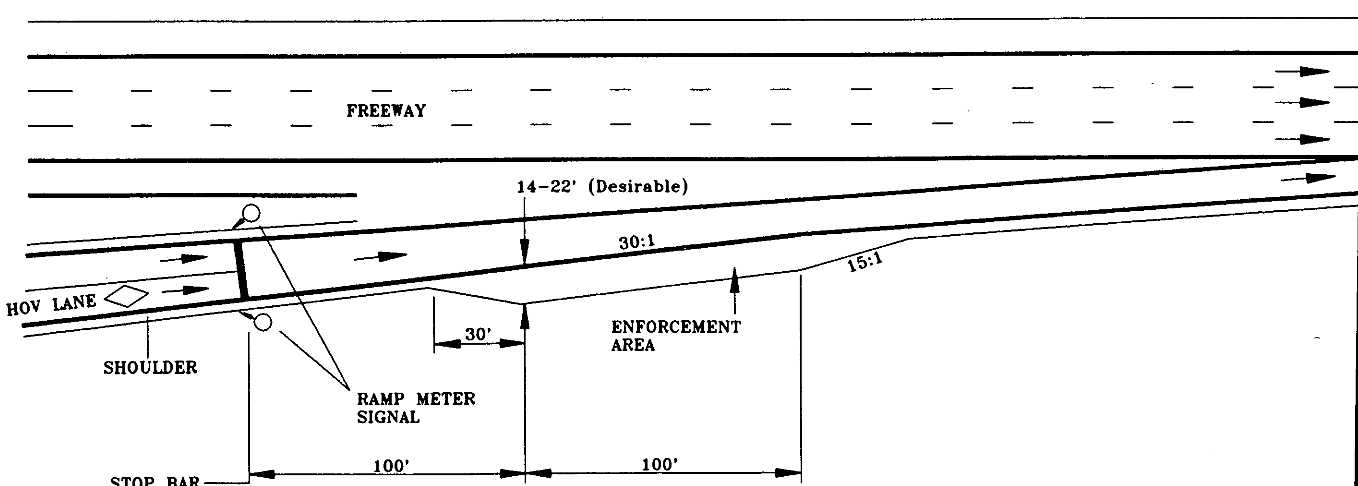

FIGURE 5 Enforcement area for HOV bypass lane at ramp meter (*2*).

Many states have constructed ramp meters and HOV bypasses in very constrained conditions, and they have worked well. This retrofit characteristic is frequently a balance between a less-than-desirable level of improvement and no improvement at all.

This paper presented a variety of designs that have been operated for some time by state DOTs that are convinced they provide a benefit to freeway operations. Some state DOTs have established rather stringent freeway operation policies that rely heavily on such metering concepts. These policies have been instrumental in widespread application of metering treatments in selected areas, and where such policies exist, geometric shortcomings are frequently accommodated in some way. Some designs work well because drivers are familiar with them and understand the behavior needed to overcome the inadequacies. A more detailed investigation of the compromises inherent in the implementation of ramp meters and HOV bypasses is needed to identify those approaches that work consistently well and those that, if installed, need more driver information to operate efficiently. That investigation should include such factors as traffic volume; number of lanes; ramp length; merge area; queue storage; signing, signalization, and marking; and ramp grade.

REFERENCES

1. Dunn Engineering Associates. *Freeway Operations Projects, North American Inventory Update*. TRB, National Research Council, Washington, D.C., 1991.
2. *Guide for the Design of High-Occupancy Vehicle Facilities*. American Association of State Highway and Transportation Officials, Washington, D.C., 1992.
3. *Guide for the Design of High-Occupancy Vehicle and Public Transfer Facilities*. American Association of State Highway and Transportation Officials, Washington, D.C., 1983.
4. *A Policy on Geometric Design of Highways and Streets*. American Association of State Highway and Transportation Officials, Washington, D.C., 1990.
5. *Urban Highway Design Procedures Manual*. Arizona Department of Transportation, Phoenix, 1989.
6. Kimley-Horn and Associates, Inc. *High-Occupancy Vehicle Bypass Lanes Location Analysis Report*. Arizona Department of Transportation, Phoenix, July 1991.
7. *Ramp Meter Design Guidelines*. Office of Traffic Operational Systems, Division of Traffic Operations, California Department of Transportation, Sacramento, July 1989.
8. *High-Occupancy Vehicle Guidelines*. HOV Systems Branch, California Department of Transportation, Sacramento, July 1991.
9. *Guidelines for Ramp Control*. Oregon State Highway Division.
10. *Guidelines for Entrance Ramp Metering*. Virginia Department of Transportation.
11. *Proposed Channelization and Striping Plans for Ramp Metering*. Washington State Department of Transportation, Dec. 1990.
12. C. A. Fuhs. *High-Occupancy Vehicle Facilities: A Planning, Operation, and Design Manual*. Parsons Brinckerhoff Quade & Douglas, New York, Dec. 1990.
13. *Manual on Uniform Traffic Control Devices for Streets and Highways*. Federal Highway Administration, U.S. Department of Transportation, 1988.
14. J. Robinson and M. Doctor. *Ramp Metering in North America*. Final report. DOT-T-90-01. Office of Traffic Operations, Federal Highway Administration, U.S. Department of Transportation, Sept. 1989.

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Interchange Ramp Geometrics—Alignment and Superelevation Design

J. A. KELLER

Several issues associated with the alignment and superelevation design of interchange ramps deserve renewed consideration in the light of research and the changing aspects of the road user. The results of recent studies and a survey of state highway design agencies are summarized. Responsible interchange design requires that the ramp geometrics be analyzed as a three-dimensional system to be certain that the facility will function as anticipated. Interchanges present the motorist with a complex set of decisions that require quick evaluation and action. Designers can reduce drivers' stress at interchanges by keeping the alignment simple and direct, maintaining design consistency, providing sight distances greater than the minimum stopping sight distances, and using above-minimum design criteria for other geometric elements. Some of the major principles that emerge include the following: (*a*) the average age of the general population is increasing, and, therefore, the characteristics of the average driver are changing; (*b*) large truck-trailer combinations are becoming more common and their requirements for proper superelevation rates and longer stopping distances exceed those of the automobile to the extent that previous margins of safety are being eroded; (*c*) spiral curves provide the most appropriate means to effect superelevation and be certain that the roadway and motorist interact in the manner expected; (*d*) superelevation rates for ramps used by large trucks should be based on reduced side friction factors; (*e*) the maximum speed differential between adjacent alignment elements should not exceed 10 mi/hr (16 km/hr); and (*f*) vertical and horizontal coordination is particularly critical when horizontal curves occur at the end of a downgrade and at the top of a vertical curve—conditions typical of interchange ramp design.

The design of interchanges concentrates most of the elements of highway design into one set of interrelated tasks with often conflicting goals. The design principles of two of these elements—ramp alignment and superelevation—should be examined in the light of changing aspects of the road user, the desire of highway design professionals to provide facilities that better meet the demands of the driving public, and recent research that explains the operational aspects of geometric features.

This paper includes information on the horizontal alignment and superelevation design of interchange ramps gathered from published reports and a 1991 survey of governmental design agencies. The information provides an improved understanding of how motorists use the highway and shows how properly designed geometrics can enhance driving and influence safety.

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FACTORS THAT INFLUENCE RAMP ALIGNMENT AND SUPERELEVATION DESIGN

The following key factors directly affect ramp alignment and superelevation design: design consistency and simplicity, the roadway user, design speed, and sight distance.

Design Consistency and Simplicity

Simple and consistent feedback must be presented to the motorist regarding the relationship between each element of the ramp geometry. Drivers' reaction time is slowed when these elements are different from those expected. Increased reaction time can be critical in an interchange area where the drivers' information processing skills are heavily taxed. Accordingly, when complex interchange designs are unavoidable, the designs should provide long sight distances; careful coordination between horizontal and vertical alignment; generous curve radii; and smooth, coordinated transitions.

The Roadway User

Many highways in the past have been designed in anticipation of use by "average" or "reasonably prudent" drivers. This has left little or no margin for the drivers whose capabilities do not meet the highway designer's expectations.

Two groups of drivers with diminished capabilities are receiving increasing attention—the elderly and those impaired by medication, alcohol, or drug usage. Truck drivers also deserve renewed consideration. Results of research on heavy truck performance show that the roadway design features that influence truck behavior need to be reevaluated.

Elderly Drivers

The increasing age of the general population will result in diminished capabilities for the average road user. Designers of interchange alignment can reduce some of the effects of increased age by increasing sight distance and simplifying interchange layout (*1*). Problems associated with the elderly driver that relate to interchange design include the following:

- Decreased visual acuity (beginning at age 40);
- Short-term memory decline, which causes problems in organizing information coming from a variety of sources;
- Reduced decision-making ability;

- Reduced ability to judge vehicle speed;
- Discomfort or pain caused by muscular stiffness and arthritis, reducing the ability to act quickly or look in all traffic directions; and
- Early fatigue, leading to slower reaction times (*2*).

Large Trucks

Since 1965 the length of the largest design vehicle in the American Association of State Highway Officials' policy on geometric design has more than doubled. Twenty-eight states out of 38 (74 percent) responding to a 1991 survey indicate that they currently consider designs for the WB-62, or larger, design vehicle. The WB-62 is a combination truck with a total length of 69 ft (21 m) and an outside minimum turning radius of 45 ft (13.5 m).

Truck operational characteristics on interchange ramps deserve additional consideration because accident records indicate that as much as 20 percent of truck accidents occur at interchanges. Traditional design guidelines for roadway alignment, based on the operation of automobiles, are not always safe for larger trucks and require modification in some instances.

Design Speed

Design speeds for ramps should bear a relationship to the type of ramp and the design speed of the connecting roadways. AASHTO recommends that ramp design speeds approximate the low-volume running speed of the intersecting highways and offers a range of speeds in Table X-1, p. 960, AASHTO *Policy* (*1*). These values apply to the sharpest or the controlling ramp curve, usually on the ramp proper, but they do not apply to the ramp terminals.

The AASHTO policy provides two sets of design criteria for horizontal curves on ramps: one set for freeway ramps and high-speed arterials and collectors [Tables X-1 and III-6, AASHTO *Policy* (*1*)] and one set for ramps connecting lower-speed arterials and collectors [Table III-17, AASHTO *Policy* (*1*)]. "Low" speed, in this context, refers to speeds of 40 mi/hr (64 km/hr) and less.

The ramp proper should be viewed as a transition area with a design speed equal to the speed of the higher-speed terminal wherever feasible. Few diagonal or loop ramps are long enough to accommodate more than two design speeds. The terminals and the ramp proper should be evaluated as a system to ascertain the appropriate speed for design.

Semidirect and direct ramps, designed to convey large volumes of high-speed traffic generally require considerable expense for their construction principally because of the grade separation structures necessary. Once it is determined that this expense is necessary, every effort should be made to apply a design speed equal to that of the connecting roadways.

Where it is anticipated that a controlled ramp will terminate in a stop sign–controlled intersection, consideration should be given to the possibility that the intersection may be signalized eventually and require geometry and sight distances appropriate for the resulting higher-speed travel.

Sight Distance

Sight distances greater than published minimums are particularly desirable at interchanges because drivers have to make judgments concerning optional travel paths, regulatory information, and often relatively severe geometrics. Accordingly, appropriate sight distances are critical to safe design because roadways with generous sight distances reduce driver anxiety and confusion, allow longer decision and maneuver times, and help compensate for driver and vehicle inadequacies.

Stopping Sight Distance

The values of stopping sight distance presented in the 1990 AASHTO geometric design policy are the minimums for avoidance of a road hazard—not leisurely stops. These minimums are based on the following conditions:

- Brake reaction time is predicated on a 2.5-sec driver reaction time, a time representative of 90 percent of the drivers in a 1971 study by Johansson and Rumar (*3*). A 3.5-sec perception and braking time has been suggested for the elderly driver (*4*).
- Braking distance values are the result of friction factors developed from studies performed from 1948 to 1962. The 1962 studies were performed on high type pavements with a locked-wheel trailer. The values were developed for wet pavement conditions. It is important to note that "truck tire braking capability is approximately 0.7 the frictional capability of passenger car tires. When a truck driver modulates the brakes to prevent spinning or jack-knifing, trucks require stopping sight distances approximately 1.4 times those of passenger cars" (*5*).
- Alignment also affects braking distance. The values provided assume that the vehicle is on a tangent section and not in a horizontal curve. Curves impose greater demands on tire friction than tangents. Therefore, the braking distance increases when the friction requirements of curves and braking are combined (*6*).
- The height of the driver's eye is set at a distance of 3.5 ft (1.07 m), whereas the object (hazard) height is considered to be 6 in. (15 cm) above the road surface. Although the extra eye height of a truck driver, 8 ft (10.4 m), helps compensate for a truck's longer stopping sight distance in some cases, the additional height does not provide an advantage against horizontal height obstructions such as bridge piers, walls, and tree foliage.

Height-of-eye distances vary, even for the automobile driver. Different automobiles position the driver's eye differently, and the eye-height of elderly drivers is lower than 3.5 ft (1.07 m) because the elderly are often shorter in physical stature (*2*).

Decision Sight Distance

Decision sight distance is important when stopping sight distance values do not provide the time necessary to process information and react appropriately. Examples of locations at interchange ramps where decision sight distance is desirable include ramp terminals at the main road, especially at an exit terminal beyond the grade separation and at left exits; ramp terminals at the cross-road; lane drops; and abrupt or unusual alignment changes.

AASHTO recognizes the difficulties presented by providing decision sight distances at the ramp exit terminals beyond a crest vertical curve (e.g., conventional cloverleaf loop ramp exit). As an alternative, AASHTO suggests the stopping sight distance plus 25 percent. Of the state design agencies responding to a 1991 survey about distances used when locating ramp exits beyond a crest vertical curve, 15 (38 percent) use the safe stopping sight distance, 9 (23 percent) use the safe stopping sight distance plus 25 percent, and 12 (31 percent) use decision sight distance. Along the ramp proper, 30 (77 percent) use the safe stopping sight distance values, and only 4 use more generous minimum criteria.

Figure 1 shows the comparative distances required for grade separations for these three sight distance conditions. The comparison assumes a 60 mi/hr (96 km/hr) design speed and an 800-ft (240-m) weaving lane centered at the structure.

Intersection Sight Distance

The sight distances provided at controlled ramp terminals should be determined in the same way as conventional at-grade intersections with consideration for the additional obstacles that are often present, such as bridge parapets, sidewalk or curb adjacent to the bridge parapet, bridge abutment or slopes, concrete median barriers, bridge piers, and retaining walls.

Both the horizontal sight triangle and the vertical sight distance must be checked to ensure that adequate intersection sight distance is available. The stopping sight distance of the vehicle on the cross-road must also be considered. In some cases vertical curves longer than the minimum will have to be designed.

Eye and vehicle elevations must be adjusted for the elevation differences in the roadway cross section and the grades of the intersecting roadways.

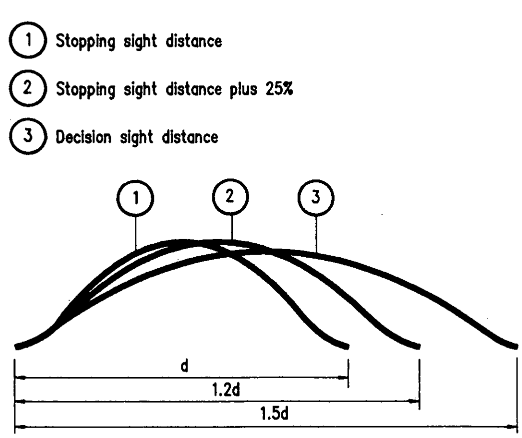


FIGURE 1 Comparative grade separation lengths for various sight distance criteria.

SUPERELEVATION

The proper superelevation rates are decided upon before beginning the ramp alignment design and after determining the traffic characteristics, selecting the design speed, and determining the basic ramp control points at the termini.

Side Friction Factors

Factors Used for Design

The maximum side friction factors (point of impending skid) developed between tires and wet concrete pavements range from about 0.5 at 20 mi/hr (32 km/hr) to 0.35 at 60 mi/hr (96 km/hr). These are not the values used for design, however. The AASHTO *Policy* states, "The speed on a curve, at which discomfort due to the centrifugal force is evident to the driver, can be accepted as a design control for the maximum allowable amount of side friction" (*1*). The margin between the maximum design value and the point of impending skid at 20 mi/hr (32 km/hr) is 0.33 and at 60 mi/hr (96 km/hr) is 0.23. The significance of this margin becomes evident when evaluating the operation of large trucks on interchange ramps.

AASHTO recognizes the special conditions that can occur at intersection curves and on ramps, such as less opportunity to develop transition lengths and the controlling geometry of adjacent lanes. On ramps with design speeds of 40 mi/hr (64 km/hr) or less, AASHTO has listed a group of superelevation rates [(*1*) Table IX-12, p. 777] with side friction factors ranging from 0.28 for a 50-ft (15-m) radius curve at 15 mi/hr (24 km/hr) to 0.02 for a 3,000-ft (900-m) radius curve at 40 mi/hr (64 km/hr).

The ramp designer is cautioned to use values in the upper half or third of the range shown wherever possible. Unless the curve is transitioned with spirals, the designer should also calculate the friction factor at the point of curvature to ensure that the suggested maximum side friction factor is not exceeded.

Factors for Large Trucks

The AASHTO friction factors are considered inappropriate for large trucks because the limiting factor in truck operation is "likely to be the rollover limit rather than skidding" (*7*).

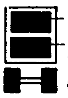
The following formula, based on a formula developed by the University of Michigan Transportation Research Institute can be used to determine the maximum friction factors for large trucks:

$$f_{max} = \frac{RT - SM}{1.15} - e_{pc} \quad (1)$$

where

RT = rollover threshold value in terms of g,
SM = safety margin = 0.10 g,
e_{pc} = superelevation at the curve PC, and
1.15 = steering factor.

Rollover threshold values and their associated truck types are shown in Figure 2.

Configuration			Rollover Threshold (g's)
	34 lb/cu ft 80,000 lbs GVW	Full gross, Medium-density Freight	0.34
	30% Wt 70% Wt 73,000 lbs GVW	Typical freight Load	0.28
	18.7 lb/cu ft 80,000 lbs GVW	Full gross, Full cube, Homogeneous Freight	0.24
	80,000 lbs GVW	Full gross Gasoline Tanker	0.32
	80,000 lbs GVW	Cryogenic Tanker	0.26

1 lb = 0.454 kg
1 in = 0.0254 m
1 lb/cu ft = 16.01 kg/cu m

FIGURE 2 Example large-vehicle configurations and rollover threshold values.

Example: A ramp terminal curve to the right is being de-
signed to accommodate a large percentage of trucks. The lesign speed is 30 mi/hr. The curve radius selected initially s 260 ft. Two-thirds of the superelevation runoff length will be placed on the tangent and one-third on the curve. The naximum superelevation rate is 0.08, and the normal cross lope is 0.02. The rollover threshold is selected from Figure to be 0.28. Determine whether the side friction factor is xcessive and whether the curve radius should be revised.

Solution: Determine e_{pc}:

$$.02 + 0.6667(0.08 - 0.02) = 0.06$$

ubstituting into Formula 1 gives

$$_{\max} = \frac{0.28 - 0.10}{1.15} - 0.06 = 0.0965$$

ubstituting this value in the standard formula for curve adius,

$$_{\min} = \frac{V^2}{15(e + f)}$$

ves a revised curve radius of 340 ft. (Note: 1 mi/hr = 1.6 m/hr, 1 ft = 0.3 m, and 15 is a U.S. conversion factor; the etric equivalent is 127.2.)

If the superelevation is developed along a spiral, the term $_c$ can drop out because the maximum superelevation will be lly achieved at the point of greatest demand.

Maximum Superelevation Rates

Values of maximum superelevation for interchange ramps are selected after accounting for the amount and frequency of ice and snow and the expected level of stopped, slow, or fast moving traffic characterized as urban or rural conditions.

Maximum ramp superelevation rates in use where snow and ice can be present are less than or equal to 0.06 or 0.08. Rates of 0.10 to 0.12 are used where snow and ice do not influence design. Urban rates are sometimes reduced to 0.06 or less. Responses to a survey of state design agencies show that the predominant superelevation rate for rural areas is 0.08 followed closely by 0.06; the predominant urban rate is 0.06. The most extreme rates reported are 0.04 (urban) and 0.12 (urban and rural).

Maximum superelevation rates for open roads are appropriate for directional and semidirectional ramps and other ramps with design speeds greater than 40 mi/hr (64 km/hr). Rates for ramps with speeds of 40 mi/hr (64 km/hr) or less are shown in Table IX-12, p. 777 of the AASHTO *Policy* (*1*). The highest rates listed in this table should be used whenever feasible.

More recent research shows that the margin of safety implied by the AASHTO side friction factors is eroded by a significant number of drivers who do not track the designed circular curve path, but follow a sharper curve path (*6*). Accordingly, the report states that more superelevation is required than is called for by AASHTO policy to produce the intended lateral tire accelerations at design speed for these drivers on an AASHTO criteria highway curve.

The designer should also consider the grades approaching the curve when selecting superelevation rates and curve radii. The steeper negative grades (downgrades) found in interchanges can lead to speeds higher than the chosen design speed—especially on loop ramps.

Superelevation Transition

Superelevation Transition Length

For ramp design, it is most convenient to determine superelevation transition lengths by establishing the relative slopes between the ramp pavement edges. AASHTO recommends values for relevant edge of pavement grades and relates them to design speed (AASHTO *Policy*, Table III-14, p. 177). These lengths roughly equal the distance traveled in 2 sec at the given speed.

Superelevation Transition Location

Traditionally, the superelevation runoff length has been located on a spiral curve leading into the circular curve. If no spiral is present, the roadway changes from requiring no superelevation (on the tangent) to requiring full superelevation instantaneously beyond the point of curvature (PC). Theory seems to favor putting most of the superelevation runoff on the tangent section so that the vehicle is positioned for the lateral forces exerted by the curved path. Current practice substantiates this rationale. Of the state design agencies responding to a survey, most (90 percent) used this general position.

With the short-length curves often encountered in ramp design, it can be tempting to begin the superelevation runoff exiting the curve immediately after the runoff entering the curve. A length of at least 50 ft (15 m) of full superelevation should be provided, however, to allow development of the vertical curves mentioned above. Without this distance, the pavement edge profile grade would change 1 percent without a vertical curve.

Superelevation Development at Free-Flow Ramp Terminals

Free-flow ramp terminals frequently join the main road in a curved alignment. The methods of developing ramp superelevation are described in the AASHTO *Policy* (*1*, pp. 966–969). The gore neutral area can be used to facilitate elevation changes between the ramp and the main road, but it should always slope away from the main line. The only exception is if the ramp joins the main road on the high side of a superelevated curve and both main road and ramp are curving in the same direction.

The maximum limits of algebraic difference between the cross-slope grades of the ramp and the main road must not be exceeded. Table IX- 14, p. 785, AASHTO *Policy* (*1*) lists these maximum algebraic differences and relates them to design speed. The most frequently cited value among the design agencies surveyed was 0.05 regardless of design speed. Other values ranged from 3 percent at 60 mi/hr (96 km/hr) to 9 percent at 15 mi/hr (24 km/hr).

The most difficult ramp terminal arrangement to design properly is one in which the ramp joins the right side of a main road curve to the left. It is difficult to achieve the proper superelevation on the ramp while not violating the crown crossover grade restrictions. This can only be resolved satisfactorily during the initial planning stages of the interchange. If the main road must curve to the left, it should be with as flat a curve as possible. For example, the radius of a main line curve with a design speed of 60 mi/hr (96 km/hr), e = 0.08, maximum, at an exit ramp with a 50-mi/hr (80-km/hr) design speed should be no greater than 4,279 ft (1289 m) or 0.75 degrees.

Superelevation and Large Trucks

Adequate superelevation enhances large truck operation, but only if the roadway is properly designed and constructed. Superelevation helps prevent truck rollover by tilting the truck in the direction opposing the lateral acceleration forces. It is not effective, however, unless it is developed early in the curve, where the truck will typically receive the highest lateral acceleration (*8*).

Locating the superelevation runoff beyond the point of curvature can result in the friction factor exceeding the AASHTO recommended maximum until full superelevation is reached, making smooth truck operation more difficult (*7*). The superelevation rate and the maximum side friction factor should be checked against the rollover threshold values for large trucks as explained in the previous section on side friction factors.

This check is particularly important at the end of long downgrades. The increase in speed often caused by downgrades should not be greater than the design speed of the curves that follow. Long, steep negative grades (greater than 5 percent) require the drivers of large trucks to be extra cautious with braking and encourage speeds above the design speed. When the ramp ends in a horizontal curve under these conditions, jackknifing or rollover becomes much more likely.

HORIZONTAL ALIGNMENT

Basic Alignment Principles

The following basic alignment principles contained in the AASHTO *Policy* (*1*) have been tailored to apply particularly to interchange ramps:

- Intersection alignment—The controlled ramp intersection angle should be as close to 90 degrees as possible, and the ramp should not intersect the cross-road at a sharp curve. The construction of short-radius curves to achieve right-angle intersections should be avoided.
- Curvature rate—The maximum degree of curvature should be used only in the most unresolvable situations, not as a means of achieving the most expedient or least costly design.
- Central angles—Large central (deflection) angles, greater than 45 degrees, are inherent in some ramp configurations, but they should be avoided where feasible. Small central angles should be absorbed by the longest curve practicable.
- Curve length—Adequate curve length is sometimes slighted, especially at small directional changes. Although the desirable curve lengths recommended by AASHTO for small central angles on open roads may not be achievable on ramps, curves must be long enough to provide proper superelevation.
- Consistent alignment—Consistent alignment, alignment expected by the driver, is facilitated by design that relates the horizontal elements to one another by design speed. The maximum speed changes between successive design elements have been recommended by various studies and design agencies. The most common value used in the United States for ramp design is less than or equal to 10 mi/hr (16 km/hr). German design guidelines for open roads recommend a change less than or equal to 6 mi/hr (10 km/hr) (*9*). Swiss design standards show a speed change equal to or less than 12 mi/hr (20 km/hr) (*10*) as being satisfactory to provide a good speed change relationship between horizontal alignment components.

Consistent alignment avoids introducing sharp curvature at or near the top of a crest vertical curve where the beginning of the curve cannot be perceived by the driver. The horizontal curve should be longer than the vertical curve. Vehicles beginning a horizontal curve near the low point of a sag vertical curve could be traveling too fast to track the horizontal curve properly.

Sharp curvature following a long tangent is not expected by drivers, who have difficulty judging speed reduction requirements and can enter the curve at too high a speed.

• Reverse curves—Reverse curves should always be separated by tangents of a length adequate to provide proper superelevation runoff. Ideally the tangent length will be long enough to accommodate both the runoff and the tangent runout. If this distance is not available, the pavement edges stay near the same elevation for a long distance, resulting in decreased transverse drainage. If the section is not long enough for the tangent runout, increase the runoff lengths until they meet in an instantaneous level section (*1*).

Drivers generally expect curves to reverse direction rather than curve in the same direction. Therefore "broken-back" curves should be avoided on diagonal ramps where such a change is not expected.

• Compound curves—Compound curves with large radii differences are inconsistent with driver expectations. It is generally accepted practice to allow the ratio of the flatter-curve radius to the sharper-curve radius to vary by no more than 2:1. If this is done, however, the arcs' lengths should be sufficient to enable motorists to decelerate and accelerate at a reasonable rate over the range of speeds.

Loop Ramps

Loop ramps have the most complicated horizontal geometrics of the ramp types mentioned. These ramps are particularly difficult because one goal is to keep them as short as possible to keep the interchange footprint small and reduce travel time. Considering these factors, AASHTO states that the practical size of loop ramp radii should be 100 ft (30 m) to 150 ft (45 m) for minor movements on highways with design speeds of 50 mi/hr (80 km/hr) or less, and 150 ft (45 m) to 250 ft (75 m) for more important movements on highways with greater design speeds (*1*).

There are three basic schemes of curve arrangement for cloverleaf loop ramps: single radius curve, flat-sharp-flat radius compound curves, and sharp-flat-sharp radius compound curves. Of these, the flat-sharp-flat combination is the most widely used, followed closely by the single radius curve. The sharp-flat-sharp combination is tempting to use because it requires less land area than the other types (for comparable weaving lengths), but it is difficult for drivers to properly judge the safe speeds required. After passing through the first curve marked with a low advisory speed, the driver may increase speed on the flatter curve to prepare for merging, being unable to perceive that the next (sharp) curve is more demanding (*11*). This speedup can be especially likely for trucks negotiating downgrades or accelerating to merge.

Spiral Curves

Spiral curves have been used effectively in highway design for decades. They have been reserved for use on high-speed roadways or by only the most conscientious designers, however, because they have been unwieldy to calculate and stake in the field.

Every motorist drives spiral curves on every roadway partly because vehicles must track a spiraled path and partly because drivers apparently desire to do so. The actual observed path emulates a true spiral (clothoid) curve of the type used in highway alignment design. The motorist then tracks a radius of circular curvature sharper than that of the road centerline before following a path generally concentric with the roadway, if the curve is sufficiently long (*6*).

Motorists are able to drive a spiraled path because the pavement is usually wide enough to accommodate such a path on a combined tangent and circular curve. If the pavement is not wide enough, lane encroachment occurs, and drivers tend to track a curve radius even more severe than described above.

Interchange ramp alignment with its relatively severe geometrics seems particularly appropriate for the use of spirals. The complexity of design and stakeout is no longer an issue. Computer-aided design and automated field survey equipment have eliminated the tedious calculations previously required.

The advantages of spirals are significant:

- Spiral curves significantly reduce side friction demand for operating speeds at or above design speeds.
- The changes in lateral acceleration and truck roll angle are smoother, requiring less driver correction.
- Spiral curves follow the driver's natural path.
- Spirals provide the appropriate location for superelevation transition.

Of the state design agencies responding to a survey, only 12 (32 percent) require spiral curves for interchange ramps. The common ramp pavement width of 16 ft (4.8 m) provides space for the motorist to drive in a spiral path of his own making, but it does not improve the superelevation distribution or reduce side friction demand.

Simple Curves

The radius of curvature (or degree of curve) is directly related to the design speed by the practical limits of superelevation rate, side friction factors, and, for large trucks, the rollover threshold. Curves sharper than the minimum radius of curvature as determined by the standard formula would require superelevation rates higher than practical or operation beyond the safe limit of tire friction.

Curve lengths should be long enough to provide the proper superelevation runoff plus a central portion at full superelevation at least 50 ft (15 m) long. AASHTO provides minimum curve lengths for compound curves on the basis of a maximum deceleration rate of 3 mi/hr (4.8 km/hr). These lengths enable the compound curve to provide a smoother transition into a sharper curve.

CONCLUSION

Successful alignment and superelevation design balances the desirable features of high design speed, long sight distance, gentle curvature, and flat grades against the constraints imposed by the natural environment, available right-of-way, social and political considerations, physical obstructions, and limited economic resources. All of these constraints push planners and designers toward using the minimum design criteria in a situation where above-minimum criteria are particularly needed. The multiple demands on drivers' attention and reaction times can be mitigated by good geometric design.

Before final design it is imperative that the designer go beyond the standard criteria and analyze the operation of each geometric element three-dimensionally in relationship to the other elements. By considering the interrelationship of horizontal alignment, vertical alignment, and superelevation and the effects they have on the driver, the designer can understand how the ramp will operate under actual conditions and how to best modify the geometrics to result in a safe, efficient design.

REFERENCES

1. *A Policy on Geometric Design of Highways and Streets*. American Association of State Highway and Transportation Officials, Washington, D.C., 1990.
2. J. Taylor. Impact of Aging Driver Population and Impaired Drivers on Geometric Design. In *Transportation Research Circular 317*, TRB, National Research Council, Washington, D.C., 1987.
3. G. Johansson and K. Rumar. Drivers' Brake Reaction Times. *Human Factors*, Vol. 13, No. 1, Feb. 1971, pp. 23–27.
4. D. A. Gordon, H. W. McGee, and K. G. Hooper. Driver Characteristics Impacting Highway Design and Operations. *Public Roads*, Vol. 48, No. 1, June 1984, pp. 12–16.
5. P. L. Olson, D. E. Cleveland, P. S. Fancher, L. P. Kostyniuk, and L. W. Schneider. *NCHRP Report 270: Parameters Affecting Stopping Sight Distance*. TRB, National Research Council, Washington, D.C., 1984.
6. J. C. Glennon, T. R. Neuman, and J. E. Leisch. *Safety and Operational Considerations for Design of Rural Highway Curves*. Report RD-36/035. FHWA, U.S. Department of Transportation, 1985.
7. R. Ervin, M. Barnes, and C. MacAdam. *Impact of Specific Geometric Features on Truck Operations and Safety at Interchanges*. Report RD-86/057. FHWA, U.S. Department of Transportation, 1986.
8. D. W. Blue and B. T. Kulgkowski. Effects of Horizontal Curve Transition Design on Truck Roll Stability. *Journal of Transportation Engineering*, Vol. 117, No. 1, Jan.–Feb. 1991.
9. *Geometric Design Standards, Guidelines for the Design of Roads*. (RAS-L-1). German Road and Transportation Research Association Committee 2.3, 1984.
10. *Highway Design Fundamentals, Speed as a Design Element*. SN 640080a. Swiss Association of Road Specialists (VSS), 1981.
11. M. Firestine, H. McGee, and P. Toeg. *Improving Truck Safety at Interchanges*. Report IP-89-024. FHWA, U.S. Department of Transportation, 1989.

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Design of Interchange At-Grade Ramp Terminals

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Ramps consist of three components: two terminal ends and a connecting roadway. The terminals can be designed as high-speed transitions between higher-function roadways or as at-grade intersections. At-grade terminal designs may be applicable at both ends of a ramp. Decisions regarding the appropriate type of terminal design are affected by the functional classification of the intersecting roadways. When an at-grade intersection is chosen as the ramp terminal design, all related controls and criteria should be based on the information presented in Chapter IX of the 1990 AASHTO geometric design policy. Operationally, when an at-grade intersection is used as the terminal, the traffic capacity analyses should be based on the intersection capacity procedures outlined in the 1985 *Highway Capacity Manual*. It is suggested that AASHTO geometric design policy emphasize the appropriateness of at-grade intersections as ramp terminals. The use of functional classification and the high-speed and low-speed design criteria for at-grade ramp terminal designs is illustrated and clarified.

t is recognized that interchange ramps are made up of three istinct components, two terminal ends and a connecting oadway (*1*, p. 958). The connecting roadway is often referred o as the ramp proper, or simply the ramp. The terminal ends hay be either free-flow ramp/freeway junctions, which typially accommodate speed-change lanes, or connecting ramp/rterial or ramp collector junctions, which may be designed s at-grade intersections (*1*, p. 978). Whereas these concepts re generally acknowledged, design criteria and controls for ne at-grade terminal ends of ramps are difficult to assimilate om the information in the AASHTO *Policy on Geometric esign of Highways and Streets* (referred to herein as the olicy). Furthermore, although issues such as functional hiarchy and design speed are implied, specific guidance rerding the application of relevant criteria to at-grade ramp rminals is not explicitly provided in the current Policy.

The objective of this paper is to identify the key design conderations of the at-grade intersection portion of an interange ramp terminal. The paper also stresses the importance design consistency and hierarchy of movement in selecting e appropriate design criteria for the connecting ramp and rning roadway. Specific design considerations of the ramp oper are provided by Harwood and Mason in this Record.

ACKGROUND

ASHTO Policy is very clear that there are three types of tersections: at-grade intersections, highway grade separations without ramps, and interchanges. It also states that "each has a field of usage in which it is practical, but the limits are not sharply defined" (*1*, p. 857). Interchanges are considered to be practical for all types of intersecting roads for any range of design speeds (*1*, p. 859).

At-grade intersections are common along arterial, collector, and local roadways as an acceptable means for moving traffic between the intersecting roadways or providing ingress and egress to adjacent land uses. The site-specific design of an at-grade intersection is an element of the overall design of a grade-separated interchange. The terminal end of an interchange ramp that forms the at-grade intersection is logically designed according to intersection criteria. Consider the at-grade ramp terminal portions of a diamond interchange. The diamond interchange type of ramp design provides the needed transition between roadways of a higher type design (e.g., freeways) and roadways of a lower type design (e.g., collector streets).

The use of an at-grade terminal design is not restricted to a diamond-type interchange. At-grade terminal design is also applicable at both ends of a ramp for a one-quadrant design along with other types of ramps that interchange traffic between an arterial and a collector, or between a collector and the on-site circulation system of an adjacent property. The current Policy certainly provides the latitude to consider these types of designs.

The intent of the AASHTO Policy is to "provide guidance to the designer by referencing a recommended range of values for critical dimensions" (*1*, p. xliii). The Policy also indicates that "sufficient flexibility is permitted to encourage independent designs tailored to particular situations" (*1*, p. xliii). Previous geometric design criteria (pre-1984 AASHTO design policies) and operational quality assessments (pre-1985 editions of the *Highway Capacity Manual*) were primarily based on traffic volume ranges. Designs performed using the earlier criteria were produced under the assumption that highways with comparable design speeds and traffic volumes would be constructed to the same standards and assumed to provide similar levels of service. It has been demonstrated, however, that there can be considerable difference in their actual operation.

Functional classification–based design criteria reflect level-of-service calculations that vary according to the function of the highway facility (*1*, p. 16). The underlying design philosophy is based on the principle of "hierarchy of movement" (*1*, pp. 1–2). Current geometric design principles begin with the establishment of the functional classification of a road segment, and then the design criteria for a road segment are

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selected to be commensurate with its function (*1*, pp. 16–17). Function is reflected by use of high- and low-design speed ranges and various measures of level of service (e.g., density, delay, reserve capacity, etc.).

Design speed is selected on the basis of factors such as topography, adjacent land use, and functional classification of the highway. The designer selects as high a design speed as is reasonable to achieve the desired degree of safety, mobility, and efficiency while considering environmental quality, economics, aesthetics, and social or political constraints (*1*, p. 63). Given these design considerations, the Policy states that "once selected, all pertinent features of the highway, including the ramps, should be related to design speed to obtain a balanced design" (*1*, p. 63).

This statement implies that if a driver has been traveling on a high-speed facility and leaves via a ramp to a lower-speed facility, the terminal end design features (i.e., curvature, superelevation, etc.) near the high-speed roadway should be based on "high-speed" design criteria. The high-speed design continues through the ramp proper until it approaches the other terminal. On the approach to the downstream end of ramps, the high-speed design would continue if the end terminal is a ramp-freeway junction. If the end terminal is a ramp/arterial or ramp/collector junction, however, a low-speed design (at-grade intersection) could be used. Likewise, once drivers enter or have been traveling on a lower functional classification roadway or street system, they have already come to accept and expect the driving environment of a low-speed road/street system. Ramps from these types of facilities could be designed using "low-speed" design criteria.

Ramp terminal criteria should be dependent on a "functional classification" for the connecting roadway. In other words, the design of the terminal ends need not be higher than the types of facilities that are being connected. The Policy supports this opinion and states that "generally, the horizontal and vertical alinement standard of ramps is below that of the intersecting highway, but in some cases it may be equal" (*1*, p. 958). This statement is the basis for the various considerations that need to be examined when selecting ramp design speed.

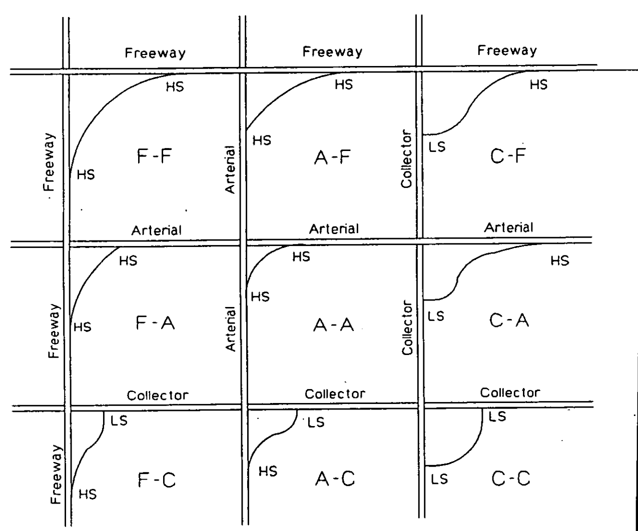
There are significant differences between design criteria applicable to low- and high-design speeds. The differences between high- and low-speed design criteria are clearly cited in the Policy. The maximum limit for low-speed design is 40 mph, and the minimum limit for high-speed design is 50 mph (*1*, p. 68). A geometric feature directly controlled by design speed selection is horizontal curvature. The parameter specifically affected by the selection of high-speed versus low-speed design criteria is f, the side friction factor, more correctly referred to as net lateral acceleration. When selecting maximum allowable side friction factors for design, the point is chosen at which the lateral force would permit a driver to experience discomfort that will, in turn, cause the driver to react instinctively to avoid higher speed (*1*, p. 143). This lateral acceleration is well below the value at which the driver would be forced "up against the door." A *g* force of that magnitude would be at the limit of available friction as when the driver is in an emergency locked wheel skid or "yawing" maneuver around a horizontal curve. In such situations, the *g* forces are, at a minimum, twice the lateral acceleration rates used in low-speed urban street design. At lower running speeds, drivers tolerate more discomfort by accepting increased lateral *g* forces. This recognition is the Policy's justification that permits a designer to use increased side friction levels for the design of low-speed horizontal curves (*1*, p. 143).

DESIGN OF THE GEOMETRIC ELEMENTS

Functional Classification

The first consideration in the design of a ramp and its at-grade terminals is the functional classification of the roadways that are being served by the ramp connector. When conditions permit, roadways are designed such that the geometric characteristics of the ramp and the at-grade terminals satisfy the "desirable" criteria recommended by AASHTO Policy. Unfortunately, constraints such as existing topography, other existing features, and cost-effectiveness often require that the design elements of an interchange be set at or near the minimum values.

To meet driver expectancy and provide a smooth transition from one roadway to another, the design characteristics of the connecting ramp and its terminal ends should be based on the functional classification of the two intersecting roadways. For example, consider the difference in the at-grade terminal and the ramp proper geometries required by Configuration F-A (freeway to arterial) as compared with Configuration F-C (freeway to connector) in Figure 1. In Configuration F-A, the ramp proper serves traffic leaving a high-speed freeway facility and entering a moderate- to high-speed arterial roadway. The ramp terminal's geometrics correspond to the applicable design criteria for the respective roadways. The at-grade ramp terminus exiting the Interstate accommodates high travel speeds. Whereas the terminus tying th



(1) HS = high-speed; LS = low-speed

(2) The use of higher design speed criteria (i.e., lower 'f' values) would be considered where topography, environment and economics permit.

FIGURE 1 Minimum design speed criteria for ramp terminals.

ramp proper into the arterial would not be designed to as great a design speed as necessary for a freeway terminus, it would still have a greater design speed than that of a terminus on a collector. Likewise, a ramp connecting an arterial roadway to a collector roadway is not designed to the same standard as a ramp connecting a freeway to a major arterial roadway.

According to AASHTO policy, the freeway is not a functional class in itself and instead is normally classified as a principal arterial (*1*, p. 15). Freeways, however, have unique geometric criteria that demand a separate design designation apart from other arterials. A roadway connecting two freeways, F-F in Figure 1, requires the use of high-speed design criteria commensurate with the geometrics of the freeway proper. The connecting roadway provides design consistency and meets the driver's expectations. It follows that driver expectancy could be violated if a "short" roadway connecting a low-speed urban street to a collector, C-C in Figure 1, roadway was designed according to high-speed design criteria. It is likely that a driver would increase operating speed through the short roadway section and subsequently be confronted with low-speed operating conditions immediately downstream.

Ramp Design Speed

Ramp design speed selection depends primarily on the function (and respective design speed) of the adjacent/connecting roadways. If the terminus end is the junction between a low-speed, low functional class roadway, the geometric elements are designed to accommodate the driver's expectancy and, as such, provide a design consistent with a low-speed design roadway. When the terminus connects with a high-speed, high functional class roadway, the design controls are selected for high-speed operations. As site conditions permit, the ramp proper provides the location to transition between geometric design requirements of the different functional roadway classes. When the functional classes of the intersecting roadways are similar, the geometry of the ramp proper is typically based on an equal or lower design speed value.

AASHTO Policy provides guide values for ramp design speeds as related to highway design speed (Table X-1, 1990). Table X-1, in the 1990 Policy, also contains the note "see Table III-6" for finding the corresponding minimum radius for each design speed. Table III-6, in the 1990 Policy, contains the minimum radii for rural highways and high-speed urban streets. These radii are based on the friction values that are considered appropriate for high-speed designs. The Policy explicitly states that these values "do not pertain to the ramp terminals, which should be properly transitioned and provided with speed-change facilities adequate for the highway speed involved" (*1*, p. 960). The Policy specifically recommends that at-grade terminal designs are "predicated on near-minimum turning conditions such as those given in Chapter IX, At-Grade Intersections" (*1*, p. 960).

Sight Distance

Providing adequate sight distance is an important element in the design of interchange at-grade ramp terminals. The Policy states that "although ramp terminals may be considered part of the interchange design . . . the terminals should be planned in accordance with design principles for at-grade intersections and with particular attention to sight distance characteristics." Therefore, intersection sight distance for at-grade ramp terminals on through roadways that are considered "low speed" in Figure 1 may be calculated using Case III—Stop Control on the Minor Roads in the Sight Distance section of the At-Grade Intersections, Chapter IX of the 1990 Policy.

The only difference concerning sight distance between at-grade intersection and at-grade ramp terminals is the added concern of the grade separation structure that supports the intersecting roadway. Where a through roadway is an underpass, the major highway abutment or columns of the grade separation structure can limit the sight distance available to the driver at the ramp terminal. In the same manner, where the through roadway is an overpass, the minor roadway parapet or bridge railing can limit the available sight distance. Figure IX-42-A, in the 1990 Policy, graphically depicts the limiting of the sight distance by the grade separation structure.

Special consideration must be taken in the design of vertical curves in the area of ramp terminals. The necessary sight distance lengths for the left-turning vehicle from the ramp terminal may make it necessary to design longer vertical curves than would be otherwise necessary to provide adequate stopping sight distance for drivers on the through roadway. According to Fitzpatrick and Mason, "K values needed to produce vertical curves that will provide the required sight distances . . . are generally greater than the Green Book K values used for vertical curve design" (*2*, p. 2). Figure IX-42-B, in the 1990 Policy, graphically depicts the possibility of the vertical curvature limiting the at-grade ramp terminal sight distance.

Ramp terminals on high-speed roadways, as shown in Figure 1, are considered free-flow facilities and have different concerns than do at-grade ramp terminals on low-speed roadways. For exiting vehicles on a high-speed roadway, sight distance must be provided to allow the vehicle to leave the through lanes of traffic without hindering the through traffic (*1*, p. 980). The sight distance considerations permit a driver to see the roadway exit (ramp terminal) and make the necessary lane changes in a safe and timely manner causing no hindrance to the through traffic. The important feature in providing this sight distance is the location of the grade separation structure for the intersecting road with respect to the free-flow ramp terminal. According to AASHTO, sight distance provided to exiting vehicles "comparable to guidelines set out under the decision sight distance discussion is recommended where practical" (*1*, p. 1033). The same sight distance consideration is necessary for the drivers of vehicles entering the roadway from a ramp terminal. Sight distance should be provided so that the entering driver is able to see the through roadway and make a well-informed decision on timing the entry into the through traffic lanes.

Certain inconsistencies are found in the sight distance recommended by the 1990 AASHTO Green Book for at-grade ramp terminals as compared with the at-grade intersection recommended values. Fitzpatrick and Mason found that the required sight distance values ("looking left"), for ramp terminal designs cited in Table IX-9 of the 1984 Policy, are 21 percent lower than the distances recommended for the same design speeds in the at-grade intersection procedure (Figure

IX-27 of the 1984 Policy) (2). The reason for this discrepancy is the use of different travel distance assumptions in each of the procedures.

The changes that occurred from the 1984 Policy to the 1990 Policy relative to recommended sight distance did not alleviate this inconsistency. Table 1 demonstrates this inconsistency. A specific explanation has not been found by the authors as to why the at-grade ramp sight distance values are different from those offered for at-grade intersections.

Horizontal Curvature

On the basis of the previous discussion, turning roadways (ramps) can fall into two distinct groups: high speed, for design speeds greater than 50 mph, and low speed, for design speeds less than 40 mph. For design speeds in the range of 40 to 50 mph, it is desirable to choose a design speed based on the higher functional classification of the connected roads for the choice of radius criteria. Low-speed design minimum radii and corresponding friction factors are found in Table III-17, in the 1990 Policy "Minimum Radii for Intersection Curves"; high-speed design radii and corresponding friction factors are found in Table III-6 in the 1990 Policy. These two tables have been consolidated to form Table 2, which can be used in conjunction with Figure 1 to select the maximum lateral acceleration values (side-friction factors) that are

1. Commensurate with the functional classification of the connecting roadways,

TABLE 1 Comparison of Recommended Sight Distance Values for P Vehicle

Assumed Design Speed on the Crossroad Through the Interchange	1984 Green Book Table IX-9[1]	1984 Green Book Figure IX-27[2] (B-1 Curve)	1990 Green Book Table IX-9[3]	1990 Green Book Figure IX-40[4] (B-2b Curve)
70	740	950	710	1550
60	630	825	610	1150
50	530	675	510	850
40	420	550	410	575
30	320	425	310	375

1 Values from column in 1984 AASHTO Green Book, Table IX-9 labeled "Sight Distance Required to Permit Design Vehicle to Turn Left from Ramp to Crossroad (ft)."

2 Values from B-1 Curve in 1984 AASHTO Policy Figure IX-27. B-1 curve is labeled in figure as follows: "Safe sight distance for P vehicle turning left into two-lane highway across P vehicle approaching from left."

3 Values from column in 1990 AASHTO Green Book, Table IX-9 labeled "Sight Distance Required to Permit Design Vehicle to Turn Left from Ramp to Crossroad (ft) and not Interfere with oncoming traffic from left."

4 Values from B-2b Curve in 1990 AASHTO Green Book Figure IX-40. B-1 curve is labeled in figure as follows: "Sight distance for P vehicle to turn left into two-lane highway and attain 85% of design speed without being overtaken by a vehicle approaching from the right reducing speed from design speed to 85% of design speed."

TABLE 2 Maximum Lateral Acceleration Values (Side Friction Factors, f) for Horizontal Curve Design at Ramp Terminals [1]

Functional Classification[2]		Minimum turning roadways			High speed merge/diverge		
	[3]	20	30	40	50	60	70
Freeway	HS	N/A	.16	0.15	0.14	0.12	0.10
	LS	N/A	N/A	N/A	N/A	N/A	N/A
Arterial	HS	0.17	.16	0.15	0.12	0.12	0.10
	LS	0.27	0.23	0.16	N/A	N/A	N/A
Collector	HS	0.17	0.16	0.15	0.14	N/A	N/A
	LS	0.27	0.23	0.16	N/A	N/A	N/A
Local	HS	0.17	0.16	0.15	N/A	N/A	N/A
	LS	0.27	0.23	0.16	N/A	N/A	N/A

[1] For horizontal curve departing or entering the adjacent roadway.
[2] From and/or to which the ramp proper is connecting.
[3] HS = high-speed criteria (Ref. AASHTO 1990, p. 154, Table III-6).
LS = low-speed criteria (Ref. AASHTO 1990, p. 197. Table III-17).

2. Consistent with the respective design speed criteria, and
3. Appropriate for the site-specific ramp terminal configuration.

The horizontal curve design controls for ramps that are recommended in the 1990 Policy are based on the criteria first presented in the 1954 *Policy on Geometric Design of Rural Highways* (*3*). Later geometric design policies, the 1957 and 1973 Urban Policy and 1965 Rural Policy, and, most recently, the 1984 Policy, serve as the foundation for the 1990 Policy (*4–7*). The following illustrates the continued consistency in horizontal curve design policy for ramps from 1954 to the present.

Table III-17 in the 1990 Policy recommends horizontal curvature for design speeds of less than 40 mph. This table is similar to Table VII-3 in the 1954 Policy and Table VII-3 in the 1965 Policy. Inspection of the three tables results in the same minimum radii for the corresponding design speed. For example, in the 1954 Policy for a "turning" design speed of 20 mph, a minimum radius of 90 ft is suggested. The recommended value for 20 mph in the 1990 Policy is also 90 ft.

The minimum radii recommended by Table III-17 in the 1990 Policy for low-speed turning roadways (less than 40 mph) are based on side friction factors that are significantly higher than the side friction factors recommended in Table III-6 of the current Policy for curves on open highways (high-speed facilities). This difference is explained by AASHTO as due to "drivers' acceptance and use of higher side friction in operating around curves at intersections as opposed to side friction accepted and used on the through highway" (*1*, p. 195).

Table X-1 in the 1990 Policy offers recommended values for ramp design speed that relate to the connecting highway design speed. The values recommended in 1990 are similar to the values originally recommended in Table IX-2 of the 1954 Policy. The similarity to Table X-1 in the 1990 Policy values is also evident in Table IX-2 of the 1965 Policy and Table J-1 of the 1973 Policy. The recommended radii (of the 1965 and 1973 Policies) corresponding to the desirable and minimum ramp design speed are based on the higher side friction factors associated with low-speed turning roadways. This method for calculating the minimum radius differs from Table X-1 in the 1990 Policy, which refers to Table III-6 in the 1990 Policy for the corresponding minimum radius. Again, Table III-6 in the 1990 Policy uses the lower side friction factors associated with high-speed facility design. This change took place when the 1965 Rural Policy and 1973 Urban Policy were combined into the 1984 version of the Policy.

Minimum-turning roadway curves for various speeds, such as those for intersection design, are presented in Chapter III of the 1990 Policy. The criteria in Chapter III apply directly to design of the curves for at-grade ramp terminals and, in some cases, the ramp proper (*1*, p. 961). The recommended radius values for minimum-turning roadways are calculated on the basis of the side friction factors cited as low-speed urban street design criteria.

The current Policy states that "three segments of a ramp should be analyzed to determine superelevation rates that would be compatible with the design speed and the configuration of the ramp. The exit terminal, the ramp proper, and the entrance terminal should be studied in combination to ascertain the design speed and superelevation rates" (*1*, p. 966). Calculation of a radius of horizontal curvature requires selection of both a "limiting" side friction factor and a "maximum" rate of superelevation as governed by local practice. The selection of the limiting side friction, however, is fundamentally determined on the basis of the functional class of the roadway and its commensurate "design speed" criteria.

The 1990 Policy states further, "On ramps designed for speeds of 40 mph or less, superelevation ranges given in Table IX-12 in the 1990 Policy are appropriate for design of the ramp proper" (*1*, p. 966). The specific radius values listed in Table IX-12 in the 1990 Policy are the results of using Table IX-12 in the 1990 Policy superelevation ranges and the respective f values of the low-speed urban street criteria.

OPERATIONAL CONSIDERATIONS

Operationally, a ramp that is designed with at-grade intersection terminals is neither a fully developed "classic" interchange nor simply an at-grade intersection. As with the geometric design aspects, discretion must be used in determining what procedures yield reasonable estimates of actual operating conditions.

Two sections of the *Highway Capacity Manual* address this type of design: the ramp capacity procedures and the intersection capacity procedures. The ramp capacity procedures are based on a set of regression equations that are used to predict Lane 1 (shoulder lane) volumes. These regression equations were developed as part of a U.S. Bureau of Public Roads (BPR) study that was done in the early 1960s. Observations were made and data recorded as part of "219 studies conducted at 195 ramp-freeway connections" (*8*). The regression equations and the techniques for using them to asses ramp terminal capacity have remained essentially unchanged through the development of the 1965 *Highway Capacity Manual* (*9*), and they are still used currently in practice from the 1985 *Highway Capacity Manual* (*10*). Whereas many of the ramp terminals that were observed had geometric features that could be considered inadequate by today's standards (i.e., no acceleration lanes or tight, intersectionlike geometry), use of these ramp capacity procedures to analyze at-grade terminals is inappropriate in most cases. The BPR procedures state that traffic coming onto the ramp "supplied via a traffic signal or an ordinary street network . . . is outside the scope of this study" and "if ramp traffic is supplied by another freeway or expressway, the 'diverging' [movement] from that facility is within the scope of this study." This attitude is echoed by the current manual, which states:

> The ramp-street junction can be of a type permitting uncontrolled merging of diverging movements to take place, or it can take the form of an at-grade intersection.
>
> This chapter [Chapter 5, Ramps and Ramp Junctions] provides procedures for the capacity analysis of ramp-freeway junctions and ramp roadways. At-grade intersections may be analyzed using the procedures of Chapter 9, Signalized Intersections, or Chapter 10, Unsignalized Intersections. (*10*)

All of these sources acknowledge that the operations of low-speed design, at-grade ramp terminal ends are comparable with at-grade intersection operations. In other words, if the terminal treatment is geometrically designed as an at-grade intersection, it follows that it should be analyzed as an at-grade intersection. Using intersection capacity procedures for low-speed at-grade ramp terminals will generally yield lower overall capacity values than the ramp capacity procedures as well.

The ramp roadway discussion in the HCM provides limited guidance concerning the operating characteristics of ramp roadways or what is termed the ramp proper. The section on ramp capacity procedures offers some guidance regarding the ramp proper in Table 5-5 of the 1985 HCM (*10*). The various conditions regarding the use of this table, however, are vague, and Table 5-5 refers only to the ramp roadway itself. Even though up to 1,700 pcph may be accommodated in a single-lane ramp, this does not guarantee that such a volume can be accommodated through a single-lane ramp terminal or at the ramp-street junction.

The HCM states that as a general rule of thumb, where volumes exceed 1,500 pcph, a two-lane ramp-freeway terminal will be needed, and a two-lane ramp should be provided. Furthermore, even where a one-lane ramp and ramp terminal are sufficient from the capacity point of view, a two-lane ramp is generally provided if the ramp is located on a steep grade or has minimal geometrics. This rule of thumb is used because with corrections for trucks and mixing traffic in the right lane, the capacity of the ramp terminal is frequently below the 1,700 value for the high-end capacity of ramp roadways. For safety and operational reasons, it is highly unlikely that there would be a two-lane exit terminal with a one-lane ramp roadway. Therefore, the heuristic is put forth as a rule that will cover a more frequently occurring situation. The HCM recognizes that this does not cover all situations and gives the designer the option of using a two-lane ramp roadway without a two-lane terminal for three reasons. They are as follows:

1. The ramp is longer than 1,000 ft, to provide opportunities to pass stalled or slow-moving vehicles.
2. Queues are expected to form on the ramp from a controlled ramp-street junction, to provide additional storage.
3. The ramp is located on a steep grade or has minimal geometric (*9*).

A brief overview of the historical development of Table 5-5 illustrates the need to carefully apply the cited guidelines. The HCM states that "there is very little information concerning the operational characteristics of ramp roadways" (*11*). Table 5-5 exists to provide guidance on "approximate service flow rates for ramp roadways" (*10*).

Table 5-5 is based on information found in a report on capacity analysis techniques for freeway facilities reported by Leisch in 1974 (*11*, p. 26). Leisch's values are slightly different from the HCM table, but it is easily recognized as the predecessor of Table 5-5. After 30 years, the values for ramp roadway capacity are still approximations without empirical basis.

SUMMARY AND CONCLUSIONS

A critical review of the historical development of interchange ramp terminal geometric design has demonstrated the following:

- AASHTO Policy clearly recognizes geometric design based on "low-speed" and "high-speed" criteria.
- The selected design speed criteria should be commensurate with the functional classification of the roadway.
- The geometric design philosophy of horizontal curve design has not changed since its inception.
- The geometric design of at-grade ramp terminals is established on the basis of the adjacent and intersecting roadway design speed.
- At-grade terminal designs forming at-grade intersections are predicated on near-minimum turning conditions of Chapter IX of the AASHTO Policy.

The review has demonstrated the following concerning operations:

- The capacity analysis techniques for ramp terminals in the current *Highway Capacity Manual* are based on observations made at ramps that were part of ramp-freeway junctions.
- The intersection capacity analysis techniques (signalized and stop control) in the current *Highway Capacity Manual* are appropriate for the analysis of at-grade ramp terminals.
- Current values for estimating the capacities of ramp roadways (the ramp proper) are based on 30-year-old approximations.

RECOMMENDATIONS

The following statements should be considered in future geometric design policies and research:

- Minimum design speed criteria should be cited for at-grade ramp terminal design on the basis of the functional classification of terminal roadway.
- The information prepared for Figure 1 could be used to clarify the use of high and low design speeds in relation to functional classification of the terminal roadway.
- A presentation similar to Table 2 could be used to combine the maximum lateral acceleration values relating to the functional classification of the terminal roadway and respective high- or low-speed criteria.
- Future research should investigate whether the allowable maximum lateral acceleration values have changed since the selection of the original values first published in the 1954 rural Policy.
- The inconsistencies between at-grade intersection sight distance values and the ramp terminal sight distance values should be examined.

In addition, the following statements are recommended for consideration regarding operational analysis of ramp terminals:

- Ramp terminals designed according to high-speed design criteria should be analyzed according to ramp junction capacity analysis procedures.
- At-grade ramp terminals designed according to low-speed design criteria should be analyzed according to at-grade intersection capacity analysis procedures.
- The capacity estimation procedures in the current *Highway Capacity Manual* should be modified to reflect the higher volumes that are frequently observed at these types of facilities

REFERENCES

1. *A Policy on Geometric Design of Highways and Streets*. American Association of State Highway and Transportation Officials, Washington, D.C., 1990.
2. K. Fitzpatrick and J. M. Mason. *Review of AASHTO Green Book Sight Distance at Ramp Terminals Procedures*. TRB, National Research Council, Washington, D.C., 1990.
3. *A Policy on Geometric Design of Rural Highways*. American Association of State Highway Officials, Washington, D.C., 1954.
4. *A Policy on Arterial Highways in Urban Areas*. American Association of State Highway Officials, Washington, D.C., 1957.
5. *A Policy on Design of Urban Highways and Arterial Streets*. American Association of State Highway and Transportation Officials, Washington, D.C., 1973.
6. *A Policy on Geometric Design of Rural Highways*. American Association of State Highway Officials, Washington, D.C., 1965.
7. *A Policy on Geometric Design of Highways and Streets*. American Association of State Highway and Transportation Officials, Washington, D.C., 1984.
8. J. W. Hess. Capacities and Characteristics of Ramp-Freeway Connections. In *Highway Research Record 27*, HRB, National Research Council, Washington, D.C., 1963.
9. *Special Report 87: Highway Capacity Manual*. HRB, National Research Council, Washington, D.C., 1965.
10. *Special Report 209: Highway Capacity Manual*. TRB, National Research Council, Washington, D.C., 1990.
11. J. E. Leisch. *Capacity Analysis Techniques for Design and Operation of Freeway Facilities*. Federal Highway Administration, U.S. Department of Transportation, 1974.

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Lengths of Left-Turn Lanes at Signalized Intersections

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The required length of the left-turn lane at signalized intersections is analyzed, and the recommended lengths for different conditions are presented. The presently available guidelines for the length of the left-turn lane are rather vague and not comprehensive. Lane lengths are analyzed from two aspects: (*a*) the probability of overflow of vehicles from the turning lane and (*b*) the probability of blockage of the entrance to the turning lane by the queue of vehicles in the adjacent through lane. The factors affecting each of these conditions are identified, and a model that computes the probability of occurrence of each of these conditions is developed. A set of tables is prepared for the recommended left-turn lane lengths for different parameters, including signal timing, left-turn volume, through volume, and threshold probabilities. Field surveys are conducted to obtain values of turning maneuver time and the space requirement per vehicle on the lane. The observed values are incorporated in the recommended lengths. The proposed models not only provide the recommended lane lengths but also help evaluate the existing conditions and the effectiveness of different improvement measures.

Suburbanization has brought about a multidirectional traffic pattern in the suburban road network. As a result, suburban intersections are handling an increasing share of turning movements. Left-turning movements, in particular, require close examination since they significantly influence the capacity, safety, and design of the intersection. The adequacy of the length of the left-turn lane, for example, affects the efficiency of movements of through as well as left-turning vehicles. This paper proposes a set of models that determines the required length of the left-turn lane at signalized intersections and presents a set of tables for the recommended lengths.

Whereas the problem is fundamental to intersection design and operation, the currently available guidelines are different among themselves, some are rather vague, and some, such as AASHTO (*1*), suggest that the length should be based on a probabilistic analysis. A brief discussion of the existing standards is provided in the next section.

In this paper, the length of the left-turn lane is analyzed on the basis of probabilities of occurrence of two cases: (*a*) overflow of the left-turn lane and (*b*) blockage of the entrance of the left-turn lane by backed-up through vehicles on the adjacent through movement lane. The models compute these probabilities as a function of left-turn volume, through-vehicle volume, signal timing (cycle length and phasing), vehicle turning time, and the length of the left-turn lane. Given an acceptable probability for each of the two cases, the required lane length is computed for that case. The recommended length is the longer of the two required lengths. The time required to complete a left-turn maneuver and the space requirement for vehicles while standing in the lane are determined on the basis of field observations.

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LITERATURE REVIEW

A literature search was conducted through HRIS and DIALOG, but no analytical model dealing with both the problems of overflow and blockage was found. The existing guidelines deal with the lane overflow problem, but they do not consider the lane blockage problem. AASHTO (*1*), however, states that blockage of the entrance to the left-turn lane should be considered when designing the turning lane. In the following, four of the existing guidelines are discussed.

1. AASHTO (*1*) suggests that the lane length should be 1.5 to 2 times the average number of vehicles that would store during one cycle.
2. NCHRP (*2*) presents a chart that is based on 2 times the average number of arrivals during one signal cycle. From a probabilistic standpoint these two guidelines do not provide a uniform standard of design. It can be easily shown that, other factors remaining constant, the probability that the lanes (designed according to the above guidelines) will overflow reduces as arrival rate increases. In other words, one would be overdesigning when arrival rates are high and underdesigning when arrival rates are low.
3. The Ontario Ministry of Transportation (*3*) offers slightly different standards. The lane length is calculated to store the average number of arrivals per cycle 95 percent of the time. In other words, the probability of lane overflow should be less than 0.05. This, although more specific than AASHTO and NCHRP guidelines, has some shortcomings. The suggested lane length is no longer valid if the existing protected phase length is less than the phase length obtained from the guideline for the existing left-turn volume and cycle length.
4. The *Highway Capacity Manual* (*4*) presents a relationship between left-turn flow rate and the required left-turn lane length based on a 0.05 threshold probability of overflow. This relationship appears also in an article by Messer and Fambro (*5*). They suggest a left-turn lane length that accounts for the average number of vehicles that remain at the end of the left-turn green phase as well as the average number of left-turn vehicles that arrive during the red time. This approach, though more comprehensive than the others, does

not include detailed analysis of lane entrance blockage by the through vehicles.

NATURE OF THE PROBLEM AND APPROACH

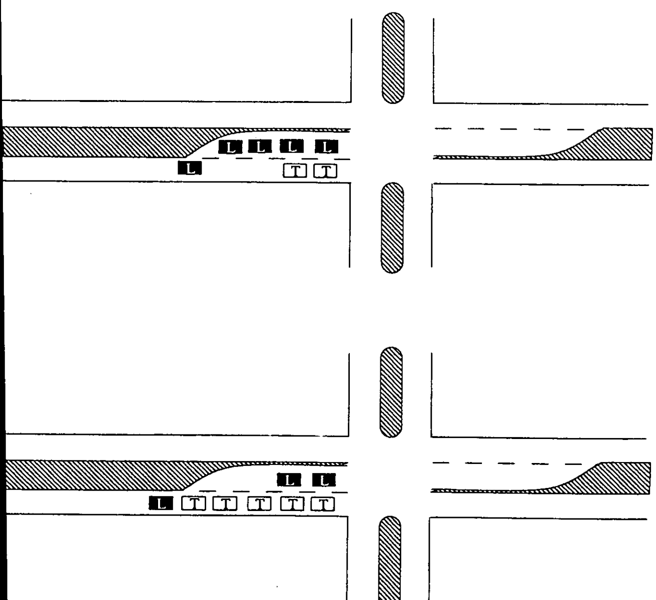
The problem is to determine the adequate length of a single left-turn lane at a signalized intersection. Inadequate lane length results in two cases: (*a*) lane overflow and (*b*) blockage of the access to the left-turn lane by the queue of through vehicles on the adjacent through lane. These situations are illustrated in Figure 1.

The causes of these cases are different. The overflow problem is heavily dependent on left-turn volume, protected phase duration, cycle length, opposing volume (if permitted phase is present), and the layout of the intersection; these factors affect the arrival and service rates of left-turning vehicles. The blockage problem is influenced more by through-vehicle volume and through red time.

Not only are the causes different, but two problems adversely affect two different populations. The overflow problem affects the smooth flow of through vehicles, whereas blockage generally increases the delay and frustration experienced by left-turning vehicles. Later, we will discuss the relative importance of these two problems.

In this paper a probabilistic approach is used to develop recommendations for left-turn lane length. The number of vehicles that arrive at an intersection and the pattern in which they arrive are assumed to be random. The implication of any probabilistic modeling is that the design is never foolproof; in other words, no matter how long the lane is, there is always a chance that the queue length will exceed the lane length. Thus, a threshold value of probability that indicates the tolerable frequency of failure is used to set the guidelines. The suggested procedure for developing the recommended length of the left-turn lane involves the following steps:

1. Determine the values of the threshold probabilities (the tolerable frequency of occurrence) of lane overflow and lane entrance blockage.
2. Identify the values of all input parameters, including cycle length, signal phasing, turning volume, through volume, and so forth.
3. Compute the necessary lane lengths by the proposed model. The lengths are given in number of vehicles.
4. Take the maximum of the two lane lengths (in number of vehicles) for the two cases (lane overflow and lane blockage) as the recommended lane length.
5. Determine the lane lengths in distance by multiplying the recommended lane length in number of vehicles by the factor that takes the length of the vehicles and the vehicle mix into account.



IGURE 1 Lane overflow and blockage of lane entrance at a ignalized intersection.

FACTORS THAT AFFECT LENGTH OF THE LEFT-TURN LANE

The major factors affecting the length of the left-turn lane are as follows:

- Traffic volumes—left-turn, through, and opposing;
- Vehicle mix—percentage of trucks, buses, recreational vehicles, and passenger cars;
- Signal timing—cycle length and phase length, protected phase, and permitted phase;
- Time required to make a left turn; and
- Space requirement for a standing vehicle (vehicle length and the gap between vehicles).

Traffic Volumes

The effect of left-turn volume on left-turn lane length is obvious. The through volume affects the lane length requirements because the queue of through vehicles on the lane adjacent to the left-turning lane may prevent left-turning vehicles from entering the lane. The opposing volume affects the requirements for the left-turn lane length if the signal phases include a permitted phase for the left-turn movement.

Vehicle Mix

The type and the mix of the vehicles influence the required length, both from the lane overflow and the lane blockage standpoints. Obviously, a large proportion of trucks will increase the probability of lane overflow for a given lane length. The probability of lane blockage also increases if the proportion of trucks in the through flow is large. Furthermore, the time required to make the turning movement differs with the vehicle type. Therefore, vehicle mix is relevant to the analysis of left-turn lane length. (In this paper, the analysis and the models are based on the number of vehicles. The vehicle mix is taken into account when the length, expressed as the number of vehicles, is converted to the actual distance.)

Signal Phases and Cycle Length

The number of vehicles that accumulate in the left-turn lane depends on the signal phases and the cycle length. It also depends on whether the left turn is protected. The number of vehicles that accumulate on the through lane depends on the duration of the red phase on the through movement. Generally, the longer the cycle length, the greater the number of vehicles that accumulate, and the longer the left-turn lane required. Thus, if the space for building the left-turn lane is limited, the signal timing should be adjusted to keep the probability of lane overflow and lane blockage below an acceptable level.

Time Required To Make a Left Turn

This time affects the maximum number of left turns that can be made during a protected phase. If T is the time required by a passenger car to complete a left-turn maneuver, RT is the perception/reaction time of the first vehicle in the queue (that is, the time gap between the signal changing to green and the first vehicle starting to move), and D is the duration of the protected green, the maximum number of left-turns, m, that can be made during D is

$$m = \text{nearest integer to } \left(\frac{D - RT}{T}\right) \tag{1}$$

The values of RT and T were determined through a field survey by video recording. The average value obtained is 2.66 sec for RT and 2.42 sec for T. These values were obtained from the following equation derived through linear regression:

$$\Delta = 2.66 + 2.423 \times (\text{total number of vehicles that made the turn}) \tag{2}$$

where Δ is the time between the onset of green and the moment when the last vehicle (that completed the left turn) initiated the turn. The R^2 value for this regression equation is 0.92. The data were collected at an intersection where the left-turn movement crosses two opposing lanes.

Space Requirement per Vehicle

The total space a vehicle requires when it is stopped on the lane is important in determining the necessary lane length. The space includes the space for the vehicle itself and the additional buffer space that a vehicle requires before it. The space required per average passenger car is estimated to be 7 m on the basis of 35 sets of observations. Queue length in the observations varied between 3 and 12 vehicles.

THE MODELS

Model for Determining the Lane Length from the Lane Overflow Standpoint

Figure 2 shows the patterns of cumulative arrivals (Line 1) and departures for the case of permitted and nonpermitted green phase (Lines 2 and 3, respectively). The queue length is represented by the vertical distance between Lines 1 and 2 and Lines 1 and 3, for permitted and nonpermitted green phases, respectively.

Assuming that the arrivals of left-turn vehicles are random, we model the fluctuation of the number of left turns in the queue at the beginning of each protected green phase (i.e., points at A, A + C, A + 2C, etc.) as a Markov process.

Assumptions

The assumptions used in the model are as follows:

1. Arrivals of left-turn vehicles are random and follow a Poisson distribution.
2. The signal phases are pretimed and thus the cycle time is constant. It includes a protected left-turn phase.
3. The maximum number of vehicles that can make left turns during the permitted phase, s, is calculated using the following equation adapted from Equation 9-22 of the 1985 HCM (*4*):

$$s = \max\left[\frac{(1{,}400 - v_o)g}{3{,}600}, 2\right] \tag{3}$$

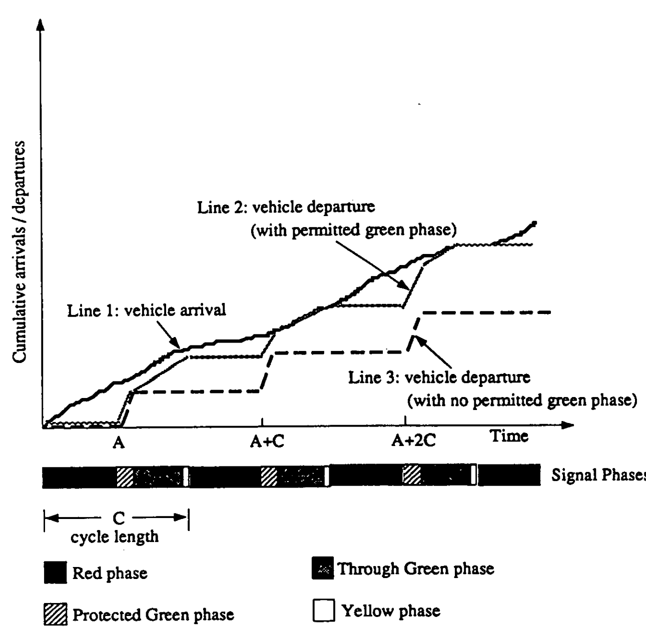


FIGURE 2 Cumulative arrival and departure processes on a left-turn lane.

where v_o is the opposing through plus right-turn volume in vph and g is the effective green time for the permitted phase.

4. The mean arrival rate of left turns is less than the capacity of the intersection to handle such movements (i.e., $\lambda_\ell C < (m + s)$, where λ_ℓ is the arrival rate of left-turn vehicles, C is the cycle length of the intersection, and m is the maximum number of vehicles that can make left turns during the protected green time). That is, the queue length will not become infinity.

5. The number of vehicles on the left-turn lane is likely to be a maximum at the beginning of the protected phase. Hence, the time points considered for the Markov process are A, A + C, A + 2C, . . . (see Figure 2). A is the beginning of the protected green phase.

Markov Chain Formulation

The system is defined in terms of states and time points. The states are the number of vehicles waiting to make a left turn at time points of analysis (i.e., A, A + C, A + 2C, . . .). The following variables are used:

- λ_ℓ is the arrival rate of left-turning vehicles in vehicles per second.
- **P** is the one-step transition probability matrix representing the probability of a change in queue length (in number of vehicles) from one time point to the next (for example, from A + C to A + 2C).
- p_{ij} are elements of **P**.
- p_α^D is the probability that α vehicles arrive during D, where D is the duration of protected green in seconds.
- p_α^R is the probability that α vehicles arrive during R, where R is the duration of protected green in seconds.
- p_α^C is the probability that α vehicles arrive during C, where C is the cycle length in seconds.
- **Π** is the vector $\{\pi_0, \pi_1, \pi_2, \pi_3, \ldots\}$, where π_i, $i = 0, 1,$ $2, 3, \ldots$, is the probability that i vehicles are waiting in the queue at one time point.
- m is the maximum number of left turns that can be made during one protected phase.
- s is the maximum number of vehicles that can turn left during one permitted phase.

Note that under the Poisson arrival with mean arrival rate of λ_ℓ, p_α^D, p_α^R, and p_α^C are given by

$$p_\alpha^\beta = \frac{(\lambda_\ell \beta)^\alpha e^{-\lambda_\ell \beta}}{\alpha!} \tag{4}$$

where β is D, R, or C.

Matrix **P** is divided into three submatrices (I, II, and III), as shown in Figure 3. Submatrix I represents the case in which the number of vehicles waiting to make a left turn (just before the start of the protected phase), i, is less than the maximum number of vehicles that can make the turn during the protected phase, m. Submatrix II represents the case in which i is greater than or equal to m but less than or equal to $m + s$, where $m + s$ is the total number of turns that can be made during the whole cycle. Submatrix III represents the case in which i is greater than the total number of turns that can be made during the whole cycle, that is, $i > m + s$. The individual elements p_{ij} of each submatrix are given below.

	0 1 . . m-1 m . m+s m+s+1 . .
0 1 . . m-1	Submatrix I
m . m+s	Submatrix II
m+s+1 . . .	Submatrix III

FIGURE 3 One-step transition matrix P.

Submatrix I applies when $i < m$. If $s > 0$,

$$p_{ij} = \begin{cases} \sum_{k=0}^{m-i} p_k^D \cdot \sum_{k=0}^{s} p_k^R + \sum_{\ell=1}^{s} \left(p_{m-i+\ell}^D \cdot \sum_{k=0}^{s-\ell} P_k^R \right) & \text{if } j = 0 \\ \sum_{k=0}^{m-i} P_k^D \cdot P_{j+s}^R + \sum_{\ell=0}^{j+s-1} p_\ell^R \cdot p_{m-i+j+s-\ell}^D & \text{if } j > 0 \end{cases} \tag{5}$$

If $s = 0$,

$$p_{ij} = \begin{cases} p_0^R \cdot \sum_{k=0}^{m-i} p_k^D & \text{if } j = 0 \\ \sum_{k=0}^{m-i} P_k^D \cdot p_j^R + \sum_{\ell=0}^{j-1} p_\ell^R \cdot p_{m-i+j-\ell}^D & \text{if } j > 0 \end{cases} \tag{6}$$

Submatrix II applies when $m \le i \le m + s$.

$$p_{ij} = \begin{cases} \sum_{k=0}^{m+s-i} p_k^C & \text{if } j = 0 \\ p_{m+s-i+j}^C & \text{if } j > 0 \end{cases} \tag{7}$$

Submatrix III applies when $i > m + s$.

$$p_{ij} = \begin{cases} 0 & \text{if } j < i - (m + s) \\ p_{m+s-i+j}^C & \text{if } j \ge i - (m + s) \end{cases} \tag{8}$$

In the following the derivation of the elements of the Submatrix I is explained using p_{i0}, the probability of finding 0 vehicles in the queue at the next time point given that there are i vehicles in the queue at the current time point, as an example.

p_{i0} = Prob{(at most $m - i$ vehicles arrive during the protected green D
and at most s vehicles arrive during the permitted phase, R),
or
($m - i + 1$ vehicles arrive during the protected green D
and $s - 1$ vehicles arrive during the permitted phase, R),

or
($m - i + 2$ vehicles arrive during the protected green D
and $s - 2$ vehicles arrive during the permitted phase, R)
or . . .
($m - i + s$ vehicles arrive during the protected green D
and 0 vehicles arrive during the permitted phase, R)}.

To calculate the limiting distribution of this Markov chain, we define an arbitrarily large number, ψ, as the upper bound of the queue length (the value of ψ is selected such that in the steady state, $\pi_\psi \approx 0$). In this case the matrix **P** becomes a $\psi \times \psi$ matrix. The elements $p_{i\psi}$ of the transition matrix now become $p_{i\psi} = 1 - \Sigma_{j=0}^{\psi-1} p_{ij}$, where p_{ij} is given by the expressions defined earlier. The conditions for **P** to be a regular transition matrix are satisfied; thus, a limiting distribution exists.

Determining the Probability of Lane Overflow

As the system reaches the steady state (in other words, the pattern of vehicle arrivals and departures is stable), the probability that a given number of vehicles exists in the lane in each time point can be computed using the following steady state equation of the Markov chain:

$$\mathbf{\Pi}\,\mathbf{P} = \mathbf{\Pi} \qquad (9)$$

where $\mathbf{\Pi}$ is a vector and each of its elements $\{\pi_0, \pi_1, \pi_2, \pi_3, \ldots, \pi_\psi\}$ represents the steady-state probability of a given queue length existing in the left-turn lane. This equation indicates that the system is the same (in terms of probability of its states) at two adjacent points. By solving the equation with respect to $\mathbf{\Pi}$, the probability of a given number of vehicles in the lane is obtained. Therefore, $1 - \Sigma_{i=0}^{N} \pi_i$ is the probability that the number of vehicles in the lane is greater than N.

Required Lane Length from Lane Overflow Standpoint

Given the tolerable probability of overflow, τ_1, the recommended length of the lane in vehicles, N^* is obtained by

$$N^* = \min\left\{N \mid \left(1 - \sum_{i=0}^{N} \pi_i\right) \leq \tau_1\right\} \qquad (10)$$

Model for Determining the Lane Length from the Lane Blockage Standpoint

This section develops a model that determines the required lane length on the basis of the probability that the entrance to the left-turn lane is blocked by the queue of through vehicles on the adjacent lane. The assumptions, formulation of the model, and calculation of the probability are presented, and they are followed by the required lane length for this case.

Description of the System and Assumptions

Given a left-turn lane length of N vehicles, the event of interest is the following: the number of vehicles in the left-turn lane is less than N, but more than N through vehicles are already waiting on the adjacent through lane, and a left-turning vehicle arrives. In this case the left-turn vehicle cannot enter the partially occupied turning lane. We will refer to this event as blockage of a left-turn lane of length N vehicles and its probability is denoted $P_B(N)$:

$P_B(N)$ = Prob{number of through vehicles $\geq N$, and the number of left-turning vehicles already in the lane $< N$, and a left-turn vehicle arrives}.

The basic assumptions made here are the same as those made for the overflow conditions. An additional assumption is that all the left-turning and through vehicles that accumulate during their red phases clear during the immediately following green phases. Also, the arrival process of through vehicles is assumed to follow a Poisson distribution with mean arrival rate of λ_t.

Model Formulation and Analysis

The probability $P_B(N)$ can be rephrased as the probability o blockage when the left-turn lane length is sufficient to stor at most N vehicles. Using the ideas of conditional probabilit we can write $P_B(N)$ as

$$p_B(N) = \sum_{L=1}^{\infty} \sum_{T=N}^{\infty} p_N^{(T,L)} \qquad (11$$

where

$p_N^{(T,L)}$ = P{blockage occurs|L turning vehicles, T through arrived} $P_L P_T$,
P_L = P {L left turns arrive during the through red R} and
P_T = P {T throughs arrive during the through red R}.

Note that the summation for T in Equation 11 is from N t ∞, because if the total number of through vehicles that arriv while the through phase is red is less than N, blockage cann occur. In the following, the probability of lane blockage derived by enumerating the different combinations of T, N and L that will cause a blockage

For $L = 1$,

$$p_N^{(T,1)} = \left\{\prod_{i=0}^{N-1} \frac{T - i}{\phi - i}\right\} \cdot \mathcal{F}(T, 1) \qquad (1$$

For $1 < L \leq N$,

$$P_N^{(T,L)} = \left\{ \prod_{i=0}^{N-1} \frac{T-i}{\phi - i} + \sum_{k=1}^{L-1} \binom{N}{k} \prod_{i=0}^{N-1} \frac{T-i}{\phi-i} \cdot \prod_{i=N}^{N+k-1} \frac{L-(i-N)}{\phi - i} \right\} \cdot \mathscr{F}(T, L) \tag{13}$$

For $L > N$,

$$P_N^{(T,L)} = \left\{ \prod_{i=0}^{N-1} \frac{T-i}{\phi - i} + \sum_{k=1}^{N-1} \binom{N}{k} \prod_{i=0}^{N-1} \frac{T-i}{\phi-i} \cdot \prod_{i=N}^{N+k-1} \frac{L-(i-N)}{\phi - i} \right\} \cdot \mathscr{F}(T, L) \tag{14}$$

where

$$\mathscr{F}(T, L) = P_T \cdot P_L = \frac{(\lambda_t R)^T e^{-\lambda_t R}}{T!} \cdot \frac{(\lambda_\ell R)^L e^{-\lambda_\ell R}}{L!}$$

$\phi = T + L$;
T = total number of through vehicles that arrive during the through red, R; and
L = total number of left-turn vehicles that arrive during the through red, R.

Required Lane Length from Lane Blockage Standpoint

Given the tolerable probability of lane blockage, τ_2, the required length in vehicles from the lane blockage standpoint, N^{**}, is calculated by

$$N^{**} = \min\{N \mid P_B(N) \leq \tau_2\} \tag{15}$$

RECOMMENDED LENGTHS OF LEFT-TURN LANE

Recommended Lengths in Number of Vehicles

Here we present a set of tables that show the required length (in number of vehicles) of a left-turn lane for different combinations of volumes and signal conditions. The required lengths are presented separately for the prevention of lane overflow and the prevention of blockage of lane entrance. Tables 1 to 3 are for the lane overflow consideration; each table corresponds to a different value of s. Table 4 is for the blockage of lane entrance consideration.

For a given set of conditions, the user first obtains the required lane lengths for the two cases separately and then adopts the greater of the two values for design. That is,

Recommended lane length in number of vehicles,

$$\text{RN}, = \max\,[N^*, N^{**}] \tag{16}$$

where N^* is obtained from Tables 1 to 3 and N^{**} is obtained from Table 4.

The tables are based on the following assumptions:

- The vehicle arrival pattern for both left-turn and through vehicles follows the Poisson distribution,
- The threshold probability (tolerable frequency of occurrence) for overflow is $\tau_1 = 0.02$, and
- The threshold probability (tolerable frequency of occurrence) for lane blockage is $\tau_2 = 0.10$.

The values of τ_1, τ_2, and other parameters are chosen to develop the tables provided here. For conditions other than the ones provided, a new set of tables should be developed using the models and considering practical and site-specific

TABLE 1 Recommended Lane Length at Signalized Intersections, Overflow Consideration: Probability of Overflow < 0.02; Number of Vehicles During Permitted Phase = 0/cycle

Left Turn Volume (vph)	Cycle Time (in seconds)															
	90				120				150				180			
	Green Time (sec.)				Green Time (sec.)				Green Time (sec.)				Green Time (sec.)			
	10	15	20	25	10	15	20	25	10	15	20	25	10	1 5	20	25
50	4	4	3	3	5	4	4	4	7	5	5	5	13	6	6	6
70	5	4	4	4	10	6	5	5	38	7	6	6	-	9	7	7
90	9	5	5	5	-	7	6	6	-	10	8	7	-	22	9	8
110	24	6	6	5	-	10	7	7	-	25	9	8	-	-	13	10
130	-	8	6	6	-	17	8	8	-	-	12	10	-	-	30	12
150	-	10	7	7	-	-	10	9	-	-	22	11	-	-	-	17
170	-	15	8	7	-	-	14	10	-	-	-	14	-	-	-	35
190	-	34	9	8	-	-	24	11	-	-	-	21	-	-	-	-
210	-	-	11	9	-	-	-	13	-	-	-	-	-	-	-	-
230	-	-	14	9	-	-	-	18	-	-	-	-	-	-	-	-
250	-	-	21	10	-	-	-	30	-	-	-	-	-	-	-	-

Note 1: "-" indicates that the required pocket becomes infinitely long for the combination of parameters.
Note 2: For conversion to length in meters under different vehicle mix, see the section on Recommended Lengths in Actual Distance.

TABLE 2 Recommended Lane Length at Signalized Intersections, Overflow Consideration: Probability of Overflow < 0.02; Number of Vehicles During Permitted Phase = 2/cycle

Left Turn Volume (vph)	Cycle Time (in seconds)															
	90				120				150				180			
	Green Time (sec.)				Green Time (sec.)				Green Time (sec.)				Green Time (sec.)			
	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25
50	2	2	1*	1*	3	2	2	2	3	3	3	3	4	4	4	4
70	3	2	2	2	4	3	3	3	5	4	4	4	7	5	5	5
90	3	3	3	3	5	4	4	4	8	6	5	5	20	7	7	6
110	4	4	4	3	8	6	5	5	24	8	7	6	-	11	8	8
130	6	5	4	4	15	7	6	6	-	11	8	7	-	28	10	9
150	8	5	5	5	-	9	7	7	-	20	10	9	-	-	15	11
170	13	6	6	5	-	12	8	7	-	-	12	10	-	-	34	13
190	34	7	6	6	-	22	9	8	-	-	19	11	-	-	-	19
210	-	9	7	6	-	-	12	9	-	-	39	14	-	-	-	37
230	-	12	8	7	-	-	16	10	-	-	-	20	-	-	-	-
250	-	20	9	7	-	-	28	12	-	-	-	35	-	-	-	-

Note 1: "-" indicates that the required pocket becomes infinitely long for the combination of parameters.
Note 2: For conversion to length in meters under different vehicle mix, see the section on Recommended Lengths in Actual Distance.
Note 3: "*" It should be noted that the values are obtained from the models for the given input values, and thus, they should be adjusted with practical and site specific considerations. In particular, the lengths less than 2 vehicles (marked by *) should not be used for most conditions.

TABLE 3 Recommended Lane Length at Signalized Intersections, Overflow Consideration: Probability of Overflow < 0.02; Number of Vehicles During Permitted Phase = 3/cycle

Left Turn Volume (vph)	Cycle Time (in seconds)															
	90				120				150				180			
	Green Time (sec.)				Green Time (sec.)				Green Time (sec.)				Green Time (sec.)			
	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25
50	1*	1*	0*	0*	2	1*	1*	1*	2	2	2	2	3	3	3	3
70	2	1*	1*	1*	3	2	2	2	4	3	3	3	5	4	4	4
90	2	2	2	2	4	3	3	3	5	5	4	4	8	6	6	5
110	3	3	3	2	5	4	4	4	9	6	6	5	24	8	7	7
130	4	4	3	3	7	5	5	5	20	8	7	6	-	12	9	8
150	5	4	4	4	12	7	6	6	-	11	8	7	-	31	11	9
170	7	5	5	4	32	8	7	6	-	19	10	9	-	-	16	11
190	9	6	5	5	-	11	8	7	-	-	12	10	-	-	35	14
210	16	7	6	5	-	17	9	8	-	-	19	11	-	-	-	20
230	36	8	6	6	-	36	11	9	-	-	37	14	-	-	-	38
250	-	10	7	6	-	-	14	10	-	-	-	19	-	-	-	-

Note 1: "-" indicates that the required pocket becomes infinitely long for the combination of parameters.
Note 2: For conversion to length in meters under different vehicle mix, see the section on Recommended Lengths in Actual Distance.
Note 3: "*" It should be noted that the values are obtained from the models for the given input values, and thus, they should be adjusted with practical and site specific considerations. In particular, the lengths less than 2 vehicles (marked by *) should not be used for most conditions.

factors. In particular, lengths less than two vehicles (marked by asterisks in Tables 1 to 3) should not be used for most conditions.

Recommended Lengths in Actual Distance

The analysis so far refers to the required or recommended lane length in number of vehicles. This value should be converted to the actual lane length in meters by taking the vehicle length and the buffer distance between vehicles into account. In this conversion the effect of large vehicles must be considered because a large vehicle not only requires a longer space but also takes more time to complete the turning movement.

The first point is obvious, and we have developed passenger car equivalency factors that account for the difference in size between a large vehicle and a passenger car on the basis of AASHTO's standard (*1*) on vehicle lengths. Their values are given in the following table:

Vehicle Type	*Symbol*	*Equivalency Factor*
Passenger car	E_{pc}	1.0
Bus	E_B	2.1
Truck	E_T	2.9
Recreational vehicle	E_{RV}	2.2

TABLE 4 Recommended Left-Turn Lane Length in Number of Vehicles, Blockage Consideration: Probability of Blockage < 0.1

Left-Turn Volume (vph)	Duration of Through Red = 45 seconds, Through Volume (in vphpl)								Duration of Through Red = 60 seconds, Through Volume (in vphpl)							
	500	600	700	800	900	1000	1100	1200	500	600	700	800	900	1000	1100	1200
50	6	7	8	9	10	11	13	14	9	10	12	13	14	16	17	19
75	7	8	9	10	12	13	14	15	9	11	13	14	16	17	19	20
100	8	9	10	11	12	13	14	16	10	11	13	15	16	18	19	*
125	8	9	10	11	13	14	15	16	10	12	13	15	17	18	20	*
150	8	9	10	12	13	14	15	16	10	12	14	15	17	19	20	*
175	8	9	11	12	13	14	16	17	10	12	14	15	17	19	20	*
200	8	9	11	12	13	14	16	17	10	12	14	15	17	19	*	*
225	8	9	11	12	13	15	16	17	10	12	14	15	17	19	*	*
250	8	9	11	12	13	15	16	17	10	12	14	15	17	19	*	*

Left-Turn Volume (vph)	Duration of Through Red = 75 seconds, Through Volume (in vphpl)								Duration of Through Red = 90 seconds, Through Volume (in vphpl)							
	500	600	700	800	900	1000	1100	1200	500	600	700	800	900	1000	1100	1200
50	11	13	15	17	18	20	*	*	13	16	18	20	*	*	*	*
75	12	14	16	18	20	*	*	*	14	16	19	*	*	*	*	*
100	12	14	16	18	20	*	*	*	14	17	19	*	*	*	*	*
125	12	14	17	19	20	*	*	*	15	17	20	*	*	*	*	*
150	12	15	17	19	*	*	*	*	15	17	20	*	*	*	*	*
175	12	15	17	19	*	*	*	*	15	17	20	*	*	*	*	*
200	12	15	17	19	*	*	*	*	15	17	20	*	*	*	*	*
225	12	15	17	19	*	*	*	*	15	17	20	*	*	*	*	*
250	12	15	17	19	*	*	*	*	15	17	20	*	*	*	*	*

Note 1: "*" indicates that the required lane length is large. A better way of dealing with the blockage problem may be changing the signal time. In most of these cases the value from this table will not be critical, since required lane length from overflow consideration will be greater.

Note 2: For conversion to length in meters under different vehicle mix, see the section on Recommended Lengths in Actual Distance.

The recommended lane length in meters, RL, is computed as follows:

$$RL = RN \times \xi \times K \qquad (17)$$

where K is the required length per passenger car and ξ is computed as follows:

$$\xi = 1 + (E_B - 1)\,\text{Prop}_B + (E_T - 1)\text{Prop}_T + (E_{RV} - 1)\text{Prop}_{RV} \qquad (18)$$

where Prop_B, Prop_T, and Prop_{RV} are proportions of buses, trucks, and recreational vehicles in the left-turning volume, respectively. The values of E_B, E_T, and E_{RV} are taken from the preceding table. The required length per passenger car, K, is found to be 7 m from the survey. This distance includes the buffer. Thus, given the vehicle mix, the recommended lane length is RN $\times\ \xi \times$ 7 (meters).

The second aspect, the effect of the longer turning time for large vehicle, requires further research. The maximum number of left turns that can be made during the protected phase, n, should decrease as the proportion of large vehicles in the left-turning volume increases. It is our belief that the value of RN will not increase by more than two vehicles because of the excess turning time of large vehicles under normal conditions.

ISSUES

This section discusses issues that must be taken into account in determining left-turn lane lengths and some issues that need further research.

Significance of Lane Overflow and Blockage of Lane Entrance

A decision on the criterion, lane overflow or blockage of lane entrance, depends on which of the movements, turning or through, is the primary flow. In most intersections, the through volume is much greater than the turning volume (although the percentage of turning movements is steadily increasing in many suburban intersections). Thus, frequent disturbances in the movement of through vehicles are not desirable. Such disturbances are caused when turning vehicles overflow into the through lane. The blockage of the entrance of the left-turn lane, on the other hand, affects mainly the flow of turning vehicles.

The basic difference in dealing with these two cases is as follows. If through vehicles are of primary importance, lane overflow becomes the more serious of the two problems, and thus this condition should be prevented. If, on the other hand, the turning movements are of primary importance, the lane blockage is the more serious problem. Thus, the threshold probabilities of overflow and lane blockage should be set according to these priorities. In developing Tables 1 to 4, primary importance has been given to the smooth flow of the through vehicles. In other words, the value of τ_1 is assumed to be much smaller than the value of τ_2 ($\tau_1 = 0.02$, $\tau_2 = 0.1$).

Selection of Threshold Probabilities

In selecting the values of the threshold probability, many factors must be taken into account, including economic, capacity, safety, and site-specific conditions. The required length is sensitive to the threshold probability; a small decrease in the value will result in a substantial increase in the length of the lane (and thus an increase in the cost of construction). A value between 0.01 and 0.02 has been used for justifying a left-turn lane at an unsignalized intersection (*6*). Both the HCM and the Ontario Ministry of Transportation guidelines use 0.05.

Comparison with the Results Obtained from NETSIM

In our model of lane blockage, we have assumed that all vehicles in the queue clear in each cycle. Reasonableness of this assumption is checked using a simulation model. For this purpose NETSIM, a network simulation model supported by FHWA, was used to simulate the blockage condition. Since NETSIM does not provide information on the probability of events, the frequency of blockage was observed by running NETSIM 100 times for a given set of traffic parameters. This was performed for 10 sets of parameters. For each of the sets of parameters, Table 5 provides the results from NETSIM of the required lane length such that the frequency of blockage is less than 10 percent. The table also provides the value of N^{**}, the required lane length from the proposed model of blockage. As can be seen, the results from the proposed lane blockage model and NETSIM are in agreement except for one case, where the difference is one vehicle.

Comparison with Existing Guidelines

The result of the proposed model is compared with those of the existing guidelines, namely AASHTO (*1*) and HCM (*4*), for the same input conditions. Figure 4 shows the suggested lane lengths for the three cases: the proposed model, HCM, and AASHTO, for different left-turn volumes. The assumed range of values for the input parameters are as follows: left-turn protected green = 10 to 25 sec, no permitted phase, cycle length = 90 sec, through red time = 45 sec, through volume = 500 to 800 vphpl, probability of overflow ≤ 0.05, and probability of blockage ≤ 0.1.

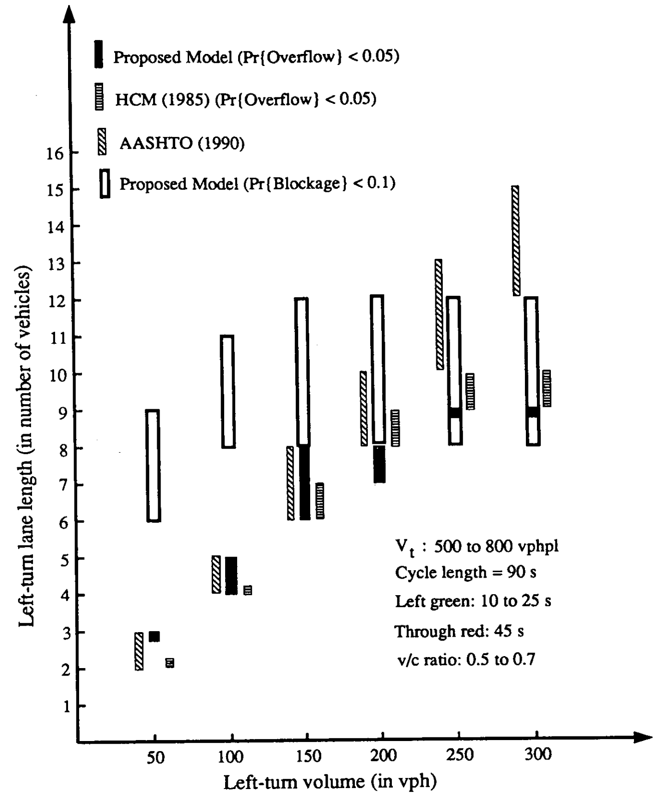


FIGURE 4 Comparison of proposed model with existing guidelines.

These assumed values of through volume and through red time are used only for our blockage model. To use the HCM guidelines, the v/c ratio for the left-turn movement is required. The v/c ratio corresponding to the above set of input parameters is between 0.5 and 0.7; thus this range of v/c is used when calculating the length on the basis of HCM guidelines.

The AASHTO and HCM guidelines are derived from the standpoint of preventing overflow of the left-turn lane only. Therefore, we first compare the result of our proposed overflow model with those of AASHTO and HCM. Figure 4 shows

TABLE 5 Comparison of NETSIM with Proposed Model Results

Spot Check Point			NETSIM Results			Proposed Model: RL
LT. Vol.	Thru' Vol.	Thru' Red	L / P	L / P	RL	
50	500	45	5 / 0.11	6 / .03	6	6
50	1000	45	10 / 0.18	11 / 0.09	11	11
90	800	45	10 / 0.1	11 / 0.06	11	11
150	1000	45	13 / 0.22	14 / 0.08	14	14
190	700	45	10 / 0.15	11 / 0.06	11	11
50	900	60	13 / 0.1	14 / 0.04	14	14
70	500	60	7 / 0.18	8 / 0.08	8	9
110	800	60	14 / 0.2	15 / 0.1	15	15
150	700	60	13 / 0.1	14 / 0.02	14	14
50	500	75	10 / 0.12	11 / 0.05	11	11

L / P : Probability of Blockage= P when lane length (in number of cars)= L
RL : Recommended lane length in number of vehicles.

Note 1: RL listed under Proposed model is the value of N^{**} obtained from Equation 15.
Note 2: RL listed under NETSIM model is the required length derived from simulation, such that frequency of blockage is less than 10%.

that the recommended length based on AASHTO is longer than that based on the other two models, and the difference increases as the left-turn volume increases. The results from our proposed model and the HCM model are close for most values of left-turn volume. However, in addition to the lane overflow, when the lane blockage is considered, the recommended lane lengths (indicated by the hollow bars) are considerably different from the existing AASHTO and HCM guidelines (which are based solely on overflow consideration).

The comparison suggests that for a small left-turn volume, attention should be paid to the possibility of lane blockage, whereas for a large left-turn volume, attention should be given to the possibility of lane overflow. This analysis emphasizes that the length of a left-turn lane is influenced by both overflow and blockage conditions.

CONCLUSIONS

This study has developed a methodology to determine the length of the left-turn lane at signalized intersections. Two cases are considered: (*a*) the left-turning vehicles from the turning lane overflow onto the adjacent through lane and (*b*) the queue of through vehicles prevents the left-turning vehicles from entering the turning lane. The parameters affecting each case are identified, and models that compute the probabilities that these cases occur are developed. A set of tables of recommended lane lengths is presented on the basis of threshold probabilities that limit the occurrence of these cases.

In addition to developing the lane lengths, the models can be used to evaluate the conditions of the existing intersections. The probabilities of lane overflow and lane entrance blockage can be computed for the existing conditions. Furthermore, alternative ways of mitigating these cases can be evaluated. For example, the appropriate signal timing, when available space for the storage lane is limited, can be analyzed.

REFERENCES

1. *A Policy on Geometric Design of Highways and Streets*. AASHTO, Washington, D.C., 1990.
2. T. R. Neuman. *NCHRP Report 279: Intersection Channelization Design Guide*. TRB, National Research Council, Washington, D.C., 1985.
3. *Traffic Control Signal Timing and Capacity Analysis at Signalized Intersections*. Ministry of Transportation, Ontario, Canada, 1989.
4. *Special Report 209: Highway Capacity Manual*. TRB, National Research Council, Washington, D.C., 1985.
5. C. J. Messer and D. B. Fambro. Effects of Signal Phasing and Length of Left-Turn Bay on Capacity. In *Transportation Research Record 644*, TRB, National Research Council, Washington, D.C., 1977, pp. 95–101.
6. S. Kikuchi and P. Chakroborty. Analysis of Left-Turn-Lane Warrants at Unsignalized T-Intersections and Two-Lane Roadways. In *Transportation Research Record 1327*, TRB, National Research Council, Washington, D.C., 1991, pp. 80–88.

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