Issues Relating to the Geometric Design of Intersections

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Introduction

This paper presents five issues, or topics, that the author suggests be addressed in the design of intersections. These are: 1) turn trajectory of vehicles; 2) the radii connecting the curb, or edge of pavement, of the two intersecting roadways; 3) intersection area; 4) the inclusion of deceleration/turn lanes; and 5) visibility of the intersection to approaching drivers.

It is also to be noted that a driveway serving private development is an intersection. Therefore, the intersection of a private driveway with a public roadway should be treated the same as the intersection of two public roadways. Moreover, the intersection of two on-site circulation roadways should be given similar attention [2004 'Greenbook', p. 729].

1. Turn Trajectory

It is commonly assumed that the design vehicle turn templates such as AASHTO provide actual turning paths. Little attention has been given to the variability in turning trajectories of driver-vehicle interaction occurring at intersections. *Transportation and Land Development* [1] presents a number of figures illustrating the dispersion of the path of right-front wheel paths for various combinations of curb return and driveway width available to drivers making a right-turn maneuver. As illustrated in Exhibit 1, the figures show the mean path of passenger cars (50% of the right-front wheel paths are to the left of this line and 50% are to the right). The figures also show plus and minus one standard deviation and plus and minus two standard deviations. Approximately 16% of the drivers of passenger cars making a right-turn are to the left of one standard deviation and 2.3% to the left of two standard deviations. Since the figures are to scale, the position of the turning vehicles can be superimposed to visualize the position of the left-side of a passenger car and, in turn, to visualize that portion of the driveway occupied, and not occupied, in the turn maneuver. Exhibit 2 illustrates the observed variability in right-turn trajectories for various driveway geometrics. These exhibits clearly show considerable variability.

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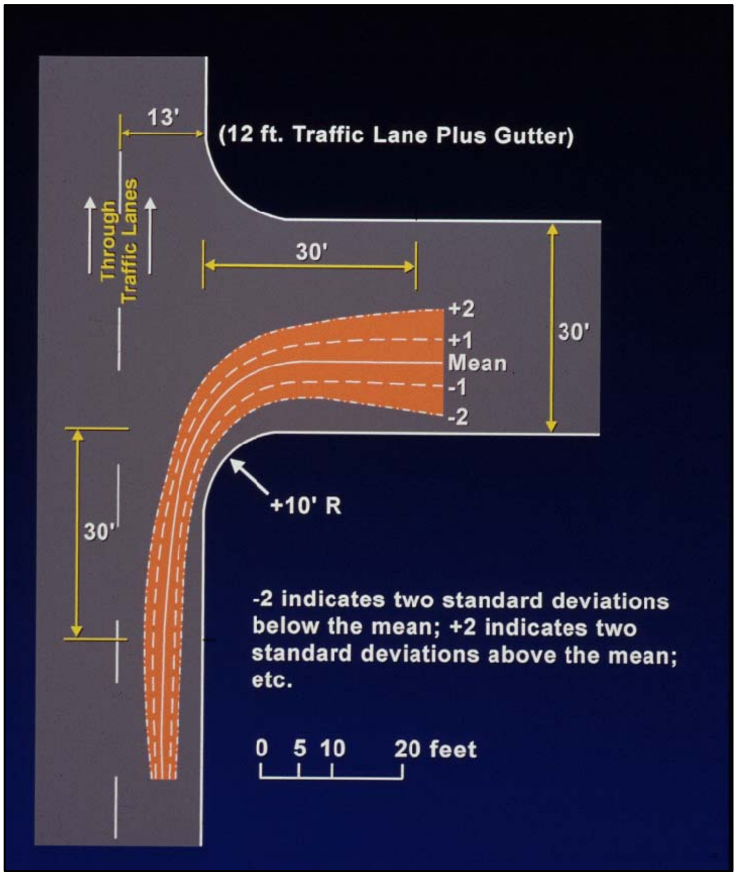
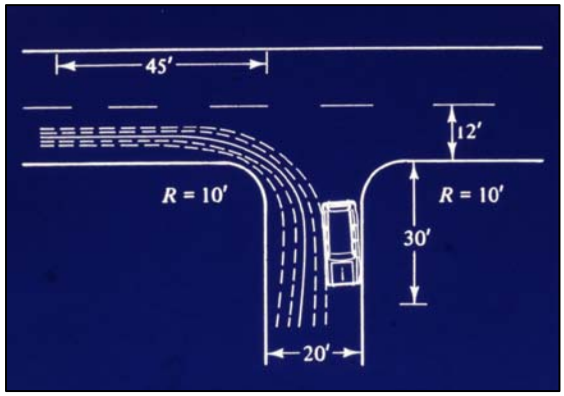
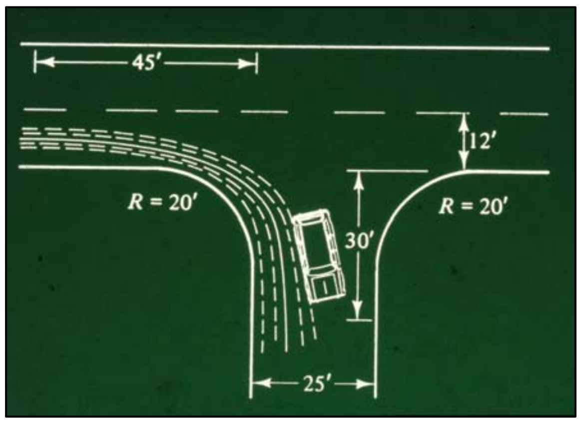


Exhibit 1: Dispersion in Path of Right Front Wheel
Source: Transportation and Land Development [1]



(a)



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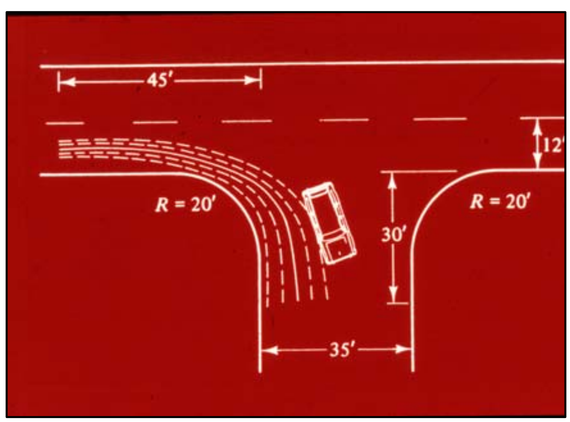

(c)

Exhibit 2: Examples of Observed Paths of Passenger Cars

Source: Transportation and Land Development [1]

The Wisconsin DOT [2] collected data for several trucks making a right-turn at two intersections. A camera system was used to obtain the observed path of the left-front overhang and the right-rear wheel of individual vehicles. Exhibit 3 shows these trajectories for one of the intersections.

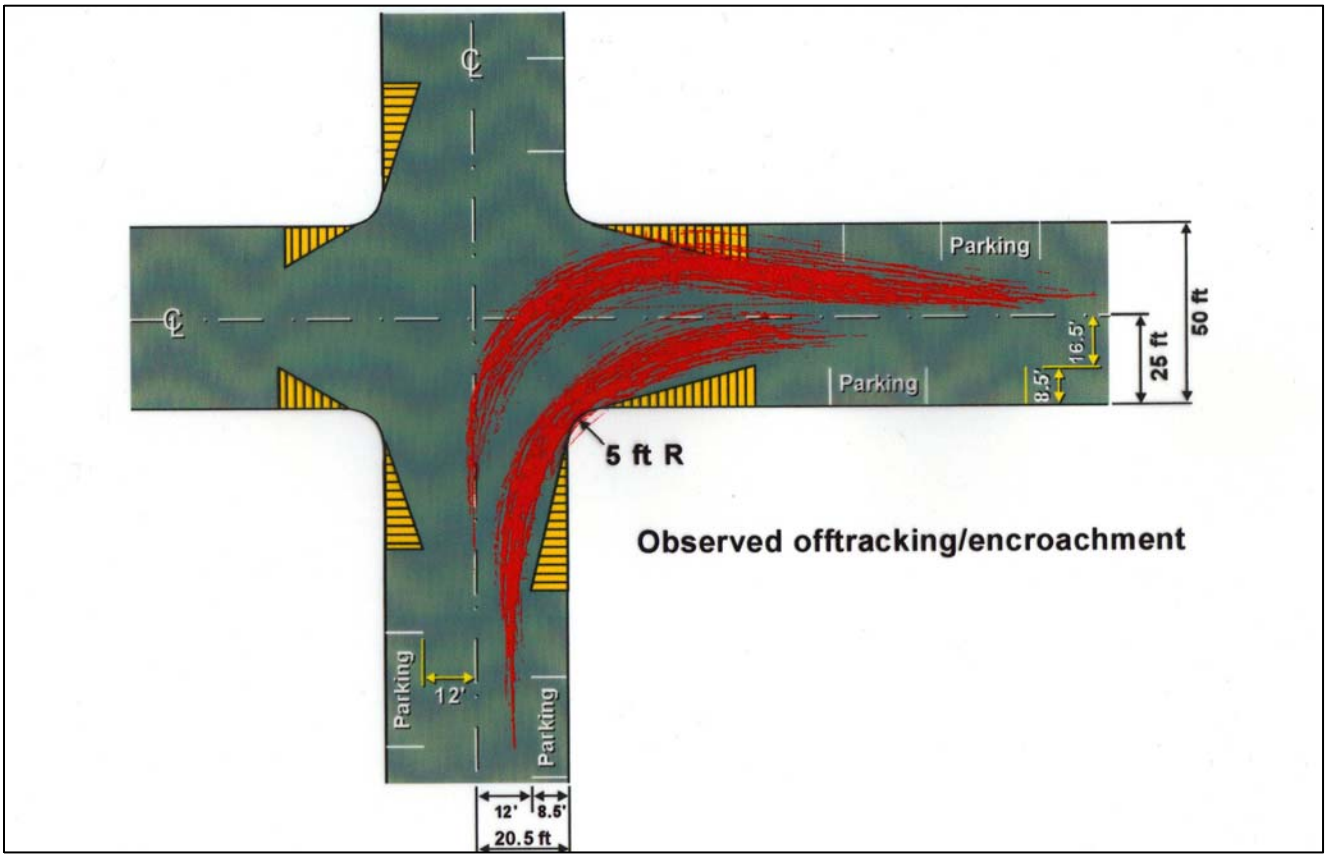

Exhibit 3: Observed Path of Trucks
Source: Wisconsin DOT [2]

Exhibit 4 compares the AASHTO templates for the SU and WB-15/WB-50 design vehicles with the average and extreme outer left-front and inner right-rear wheel trajectories. Inspection of this figure shows that an SU vehicle can make the right-turn without physically encroaching into the opposing traffic lane and the WB-15/WB-50 vehicle encroaches only slightly. Whereas the average left-front overhang of all trucks occupies the entire opposing traffic lane and the extreme outer path (approximately the 95th percentile) encroaches upon the parking lane. (Note that parking is prohibited near the intersection to provide for this encroachment.)

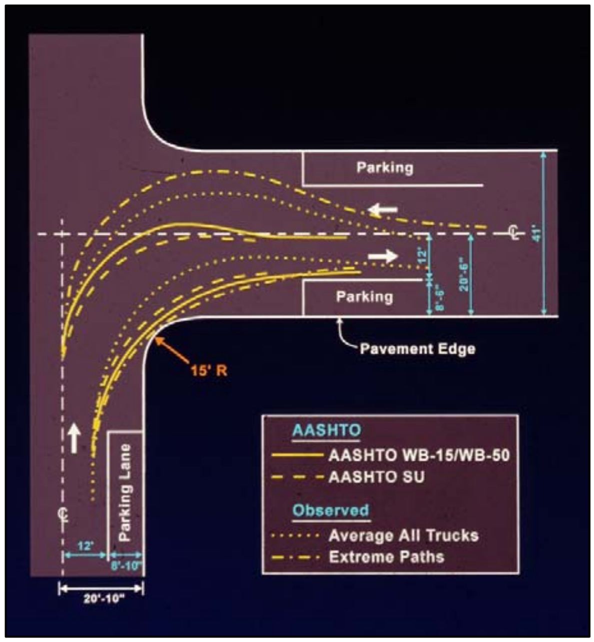

Exhibit 4: Comparison of Observed Turn Trajectories and Turn Templates

Research by the Mack-Blackwell Transportation Center [3] reported that: 1) The turning path templates do not present the rear kick out observed at the beginning of a turn (they noted that this outside rear path of the bus body may represent a controlling factor, and 2) the large over steer of actual school buses varies from drive-to-driver and is difficult to represent with turning path models. (This research did not however, provide data as to the extent of the dispersion in turning paths.) Therefore, even though currently accepted turning path models, such as computer simulation, represent the turning path of school buses, their direct use for intersection design is questionable.

2. Radii

Logic suggests that the intersections that serve truck routes, as well as warehouse locations and other areas where frequent truck traffic can be expected should be designed for large trucks (at least the WB 19/WB-62). Intersections in urbanized areas should be designed to readily accommodate transit buses.

An inappropriately short radius will require a right-turning vehicle to 1) encroach upon the adjacent traffic lane when making the turn; or encroaching upon the opposing traffic lane, or using the entire width of the receiving roadway; or both, or, 2) off-tracking of the right-rear wheels. Exhibit 5 shows a location where right-turning trucks off-track over the curb resulting in physical damage to the intersection curb return and the pedestrian sidewalk.

Exhibit 5: Inadequate Radius Results in Damage to Curb Return and Sidewalk

Exhibit 6 shows a commercial driveway where the radius was increased to enable a passenger car driver to make a right-turn maneuver into the curb-lane.

Exhibit 6: Access Connection Was Reconstructed to Accommodate the Physical Characteristics of a Right-Turning Passenger Car

Where on-street parking is permitted, a "bulb-out" reduces the unprotected distance a pedestrian must walk when crossing a street. However, the radius must be suffici9ent to accommodate the right-turn by the appropriate design vehicles.

Exhibit 7: Radius Must Accommodate the Physical Limitations of the Selected Design Vehicle

Minimum Curb Return Radii				
	Number of Receiving Lanes			
	1		≥2	
Design Vehicle	metres	feet	metres	feet
Passenger car	6.1	20	3.1	10
SU	10.7	35	7.7	25
City Bus	10.7	35	7.7	25
WG-19 WB-62	12.2	40	9.2	30

*vehicle turning right from the curb lane utilizes at least 2 lanes of the receiving roadway

Surface drainage at an intersection presents challenges, especially where the terrain is relatively "flat". Exhibit 8 illustrates a situation where the very limited longitudinal profile and the absence of a storm water sewer resulted in a "bird bath" within the pedestrian cross-walk.

Exhibit 8: Poor Intersection Design Resulted in a Drainage Problem

3. Intersection Area

The term intersection is commonly used in reference to the physical area as determined by the return radii connecting the edges of the intersecting roadways – including marked or unmarked pedestrian cross-walks. From an operational point of view, the functional intersection area extends some distance upstream and downstream from the physical intersection as illustrated in Exhibit 9. The upstream distance is comprised of the following three elements (See Exhibit 9): 1) distance traveled during a perception-reaction time; 2) a deceleration-maneuver distance, and 3) a queue storage distance. The deceleration-maneuver distance may be established by the distance required to decelerate to a stop or by the impact distance (the distance upstream at which the brake lights are activated in response to a preceding turning vehicle or stopped queue in the traffic lane.

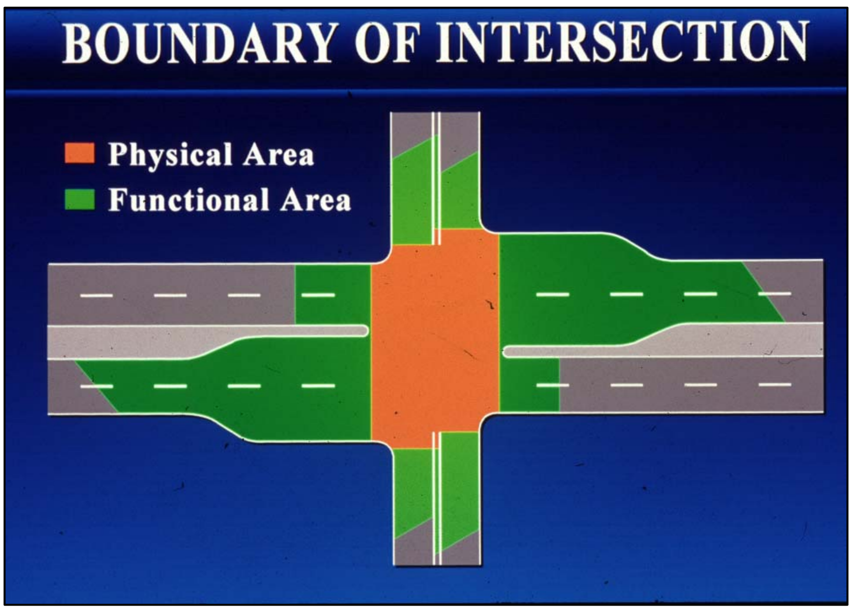

Exhibit 9: Schematic Illustration of Physical and Functional Intersection Areas

Exhibit 10: d_1, Distance Traveled During Perception-Reaction Time

Customary US Units[1]				Standard International Units[2]			
Speed (mph)	Distance-Feet, for Perception-Reaction time at:			Speed (km/h)	Distance- Metres, for Perception-Reaction Time at:		
	2 sec.	3 sec.	4 sec.		2 sec.	3 sec.	4 sec.
30	90	130	175	40	22	33	44
40	115	175	235	50	28	42	56
50	145	220	295	60	33	50	67
60	175	265	355	70	39	58	78
70	205	310	410	80	44	67	89
				90	50	75	100
				100	56	47	111

(1) Rounded to 5 ft.
(2) Rounded to 1 m

Exhibit 11: d_2, Deceleration-Maneuver Distance

Customary US Units		Standard International Units	
Speed (mph)	Deceleration-Maneuver Distance, feet[1]	Speed (km/h)	Deceleration-Maneuver Distance, metres[2]
30	160	40	30
40	275	50	50
50	425	60	75
60	610	70	100
70	820	80	130
		90	165
		100	205

(1) 5.8 fps^2 deceleration while decelerating and moving laterally, 6.7 fps^2 deceleration thereafter;
(2) 1.8 m/s^2 deceleration while decelerating and moving laterally;

Source: Transportation and Land Development [1]

Exhibit 12: d_3, Example Queue Storage Lengths[(1)]

Left-Turn Volume (vph)	Cycle Length (Seconds	Customary US Units (feet)		Standard International Units (metres)	
		Single Lane	Dual Left[(2)]	Single Lane	Dual Left[(2)]
50	90 120	75 100	-- --	20 25	-- --
100	90 120	125 175	-- --	40 50	-- --
150	90 120	200 250	100 150	60 75	40 45
200	90 120	250 350	150 200	80 100	45 60
250	90 120	-- --	175 250	-- --	55 75

(1)/Storage Length L = V/N ks

Where: V = left-turn volume per hour

N = number of cycles (time intervals per hour)

= (3600 sec/hr.)/cycles, time intervals, per hour

k = 2.0; storage length to accommodate the longest queue approximately 95% of the time (time intervals)

s = 2.5 ft. (7.6 m) storage per vehicle assumes ≤5% large vehicles. For large vehicles > 5% of left-turn volume, multiply tabled values by:

% large vehicles	Adjustment factor
10% 15% 20%	1.25 1.35 1.50

(2)Dual left-turn lanes are suggested when the left-turn volume exceeds 200 vph.

The downstream functional distance is, at a minimum, stopping sight distance. AASHTO assumes an average deceleration rate of 11.2 fps^2 (3.4 m/s^2) and 2.5 second perception-reaction time [2, 2004 ed., pg. 112]. These distances, given in Exhibit 13, are optimistic in view of the assumed deceleration rate. However, an alert driver may require less perception-reaction time.

Exhibit 13: AASHTO Stopping Sight Distances on Level Grade

US Customary Units		Standard International Units	
Speed mph	SSD (feet)	Speed km/h	SSD (metres)
30 40 50 60 70	200 305 425 570 730	40 50 60 70 80 90 100	50 65 85 105 130 160 185

The ideal minimum spacing of unsignalized access connections is the sum of the downstream functional distance of an access connection plus the upstream functional distance of the next connection as illustrated in Exhibit 14. This allows the adjacent access connection to operate without negatively influencing each other. **Corner clearance is a special case of access connection spacing.** Thus, the distance between a signalized intersection and the nearest upstream unsignalized access connection (driveway) and an intersection should be the sum of the downstream functional distance of a driveway (in Exhibit 14) and the upstream functional distance of an intersection (B in Exhibit 14). Thus for example, the minimum separation between a driveway and a subsequent intersection (upstream corner clearance of the intersection) on a 40 mph roadway, assuming a 2.0 second perception reaction time and 175 queue storage (d_3 in Exhibit 9) is:

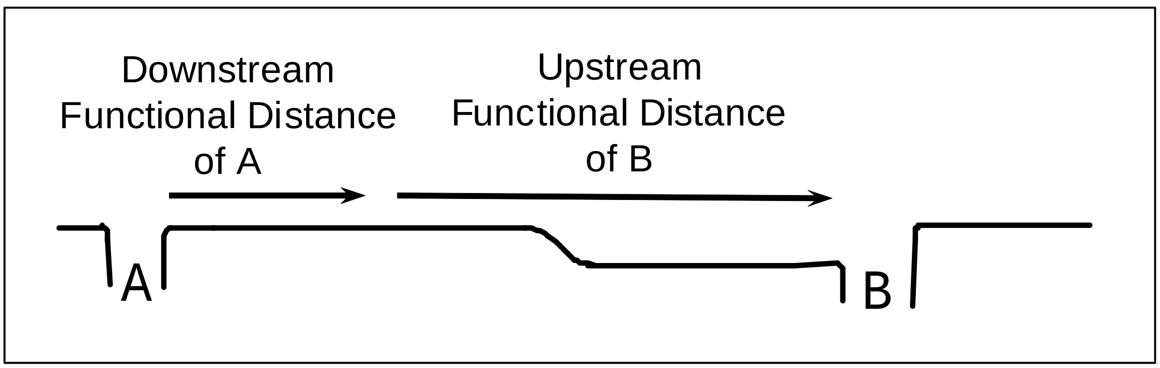

Exhibit 14: Ideal Access Connection Spacing

Downstream functional distance of driveway =			305 ft.
Upstream function distance of intersection:			
d_1 (2.0s perception-reaction)	=	115 ft.	
d_2	=	275 ft.	
d_3	=	175 ft.	
			565 ft.
Minimum Separation = (distance from near edge of driveway to near edge of intersection)			870 ft

The same criteria should be applied to the separation between an intersection and a downstream driveway. The difference is that the site served by the driveway should be designed so that right-turn queue storage is provided on-site. The separation from the conditions assumed above is:

Downstream functional distance of driveway =			305 ft.
Upstream function distance of driveway:			
d_1 (2.0s perception-reaction)	=	115 ft.	
d_2	=	275 ft.	
d_3	=	-0- ft.	
			390 ft.
Downstream functional distance of intersections			695 ft.

Corner clearance is a special case of access connection spacing. And, minimum corner clearances should not be less than minimum spacing standards. The Colorado State Highway Access Code recognizes this by setting minimum spacing based on speed with no separate mention of corner clearance. The Missouri DOT guidelines also recognize this by including a table of minimum spacing and a table of minimum corner clearance – both of which contain the same values.

In specific circumstances it will be necessary to deviate from adopted spacing standards. It is essential that an agency's regulations include written criteria and procedures for addressing a request for deviation from spacing standards. Corner clearances are illustrated in Exhibit 15. The following are suggested as absolute minimum distances.

- A, upstream from an intersection: upstream functional distance (d_1, +d_2 + d_3); in no case less than the physical length of a right-turn bay plus 25 ft.

- B, downstream from an intersection: desirably the upstream functional distance (d_1 + d_2) of the driveway (the site access and circulation should accommodate queuing on-site; therefore d_3 will normally be zero); in no case less than stripping sight distance if no right-turn lane is present or the length of the right-turn bay serving the driveway (including taper) plus 25 ft. where a right-turn lane is present.

- C, upstream on the minor cross-road: largest expected queue.

- D, downstream on the minor cross-road:
  - a) $\geq$120 ft. if unchannelized
  - b) $\geq$230 ft. if radius is 75 ft.
  - c) $\geq$275 ft. if radius is 100 ft.

❖ Intersection of two major streets:
   a) upstream on all approaches: same as A above
   b) downstream on all approaches: same as B above

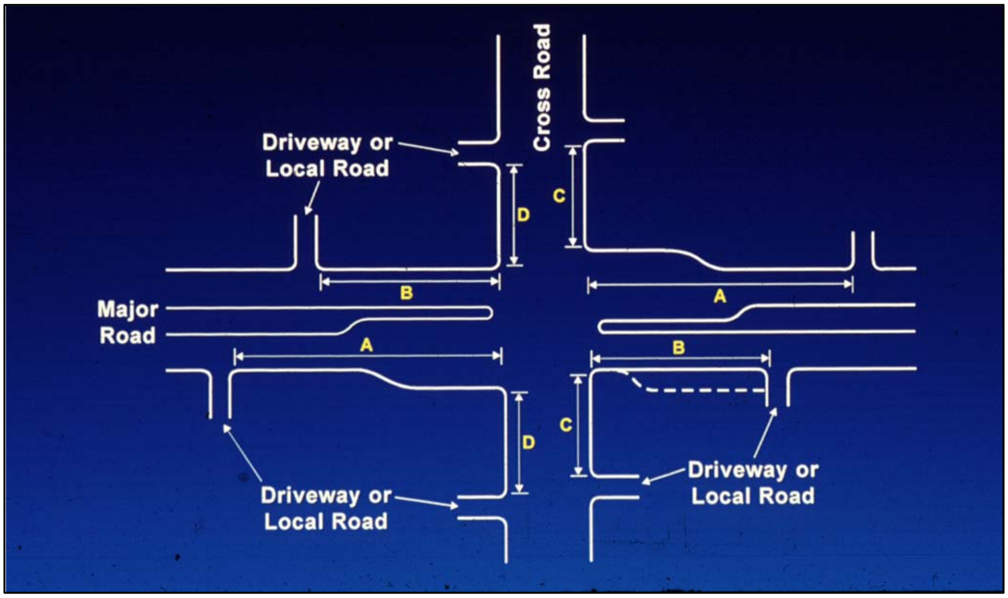

Exhibit 15: Corner Clearance

4. Deceleration Lanes

All intersection and driveway geometrics result in right-turning vehicles making the maneuver at slow speed [1]. Therefore, the speed differential between a vehicle making a right-turn from a through traffic lane and following though vehicles is essentially the speed of traffic. As shown in Exhibit 16 the likelihood of a collision increases exponentially as the speed differential increases.

Exhibit 16: Relative Crash Rates on At-Grade Arterial					
	Speed Differential (mph)				
	0	-10	-20	-30	-35
Ratio, 0-mph differential 10-mph differential	1	2 1	6.5 3.3	45 23	180 90

The data indicate that a vehicle traveling on an at-grade arterial at a speed of 35 mph (56 km/h) than the average speed of the traffic stream is 180 times (20,000/110) more likely to be involved in an accident than a vehicle traveling at the average speed. A vehicle traveling 25 mph (56 km/h) slower than the other traffic has about 90 times (20,000/220) the change of being involve din a crash as a vehicle going only 10 mph (16 km/h) slower.

While the relative rates may be in error for any specific section of roadway, they clearly show the increased accident potential.

The only practical way to limit the speed differential is to provide a right-turn deceleration lane. Exhibit 17 indicates that interference to through traffic increases substantially when the volume in the right lane exceeds about 350 vph.   Investigations by Hawley and Stover [4] found that "cut-offs" of 325 vph at 55 mph, 350 vph at 45 mph, and 375 at 35 mph. These results indicate that the real problem is right-lane volume, not right turn volume.

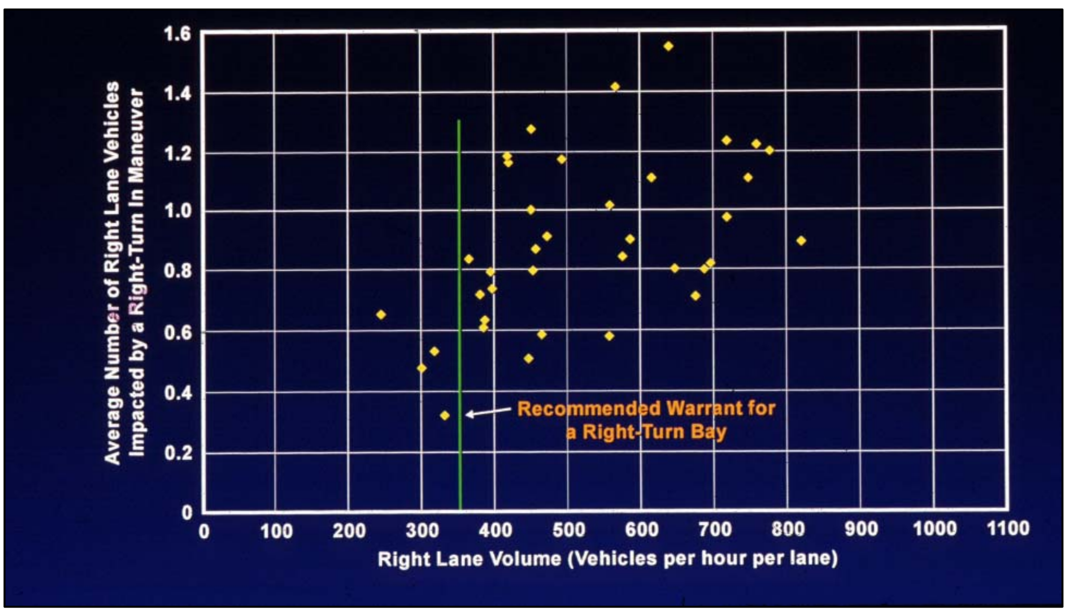

Exhibit 17: Vehicles in Right Lane Impacted by a Right-Turning Vehicle

It needs to be recognized that a vehicle making a right-turn from the through traffic lane will disrupt platooned flow. Vehicles following the turning vehicle will not be able to maintain progression and will be stopped at the next signalized intersection. Therefore, in addition to long and uniform signal spacing and an effective signal turning plan, right-turn lanes are needed to obtain efficient traffic progression.

Left-turn lanes are also essential in order to maintain traffic progression, reduce delay, minimize fuel consumption, and limit vehicular emissions. Recent investigations [4, 5, 6, 7] have Harmelink's assumptions (and consequently Harmelink-based warrants such as AASHTO's) are not correct and that the warrants for left-turn lanes should be much lower than that used by most agencies (NCHRP Project 03-91 will research this issue.)

5. Visibility

A driver must be able to locate an access connection and determine its geometrics in order to maneuver safely. This is a decision sight distance issue (DSD) and involves longer distances and different height of object criteria than stopping sight distance (SSD) or intersection sight distance (ISD).

Determination of the geometrics of an intersection requires that the driver see the pavement surface (zero height of object). Determination as to location or minor intersections (public roadways or private drivers) also requires that drivers be able to see the pavement surface. Information as to the location of major intersections (including the access drives of large traffic generators) can be provided so that drivers obtain the location information when they are some distance from the access connection and then obtain the geometric information when they are very close.

Traffic signals provide location information of major urban/suburban intersections. Additional location information can be provided by large, internally illuminated signs mounted on the traffic signal mast arms. The use of larger signs (1.5 to 2.0 times that commonly used as shown in Exhibit 18) would enable drivers a greater distance and would be especially helpful to older drivers. Advance street name signs can also be used to provide intersection location information (Exhibit 19). Properly located signs, and lighting, can identify the location of the access drives of large private development.

Exhibit 18: Example of Mast Arm Mounted Sign to Aid in Intersection Visibility/Identification

Exhibit 19: Example of Advance Information Sign Informing Drivers of an Intersection

A practice of some municipalities is to include:

"Next Signal" to the advance information sign to provide further information.

Landscaping also provides information as to location as drivers near an intersection and accentuates the geometric features.

In rural areas, mounting a roadway name plate under the standard black-or-yellow intersection warning sign has been used to provide location information.

References:

1. V. G. Stover and F. J. Koepke, Transportation and Land Development, Institute of Transportation Engineers, 2nd edition, 2002.

2. P. H. Decabooter and C. E. Solberg, Designated Highway System Truck Operation Study, "Geometric Considerations", presented at the 67th Annual Meeting of the Transportation Research Board, January 13, 1988.

3. J. L. Gattis and M. D. Howard, Large School Bus Design Vehicle Dimensions, Mack-Blackwell Transportation Center, University of Arkansas, September 1998.

4. P. E. Hawley and V. G. Stover, "Guidelines for Left-Turn Bays at Unsignalized Intersections", Second National Access Management Conference, August 11-14, 1996, pp. 383-391.

5. L. K. Staplin, K. Lococo, S. Byington, D. L. Harkey, Highway Design Handbook for Older Drivers and Pedestrians, Report No. FHWA-RD-01-103, Federal Highway Administration, 2001.

6. K. Fitzpatrick and T. Wolff, "Left-Turn Lane Installation Guidelines", Compendium of the 2nd Urban Street Symposium, Anaheim, California, July 26, 2003.

7. Ida VanSchalkwyk and V. G. Stover, "Harmelink Revisited", 3rd Urban Street Symposium, Seattle, Washington, June 25-26, 2007.