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DECELERATION LANES ON LEFT-TURN BAYS OF FOUR-LANE EXPRESSWAYS

Final Report

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16. Abstract

The overall objective of the research project was to develop guidelines for determining whether an offset left-turn lane (Type A) or a traditional left-turn lane (Type B) is more cost-effective on rural unsignalized four-lane expressways in Nebraska and to determine desirable geometric characteristics for the left-turn lane design for high-speed through traffic conditions.

Data collected showed that while offset left-turn lanes provide enhanced safety by providing an uninterrupted sight line beyond an opposing left-turning vehicle, they decrease safety due to left-turners decelerating significantly in the through passing lane before entering the offset left-turn bay.

Costs used in the analysis method were construction, maintenance and operating, traffic operations and safety related. The operational effects of left-turn lanes were determined by means of computer simulation to evaluate the effects on vehicle delay and fuel consumption and develop regression equations based on traffic conditions and available storage length of the left-turn bay. Safety effects were assessed in terms of the expected accidents derived from 1) speed differences between left-turning and through vehicles and 2) the advantage of the uninterrupted sight line of offset left-turn lanes. This research project developed a spreadsheet procedure using existing Nebraska Department of Roads design policies to determine the lowest cost left-turn lane to construct given the known variables of: expressway and crossroad AADT, percent lefts and heavy trucks on the approach, equivalent left-turn storage length, approach grade, paved area of left-turn lane, expected posted speed limit, type of project (large or retrofit), material type (concrete or asphalt) and lighting.

Recommendations were made to revise current NDOR left-turn lane geometrics to encourage drivers to decelerate in the left-turn taper rather than in the through passing lane.

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DISCLAIMER

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Background

Left-turn lanes are provided at median openings for access to intersections, residential or commercial driveways, and field entrances on rural four-lane expressways constructed with 40 ft median widths in Nebraska. The left-turn lane geometric policy (*1*) currently being used by the Nebraska Department of Roads (NDOR) designates two types of designs for those locations with a high likelihood of multiple turning movements per day:

1) Type A Median Break – Offset Left-Turn Lane (Figure 1.1): This design shifts the left-turn storage lane laterally in the median toward the opposing traffic side to improve sight distance normally restricted by opposing left-turning vehicles. Other benefits of this design include increased storage length, reduced distance to complete the left-turning movement and decreased possibility of conflict between opposing left-turn movements within the intersection. This type of design is used primarily at expressway intersections with major state highways, and high-volume county roads and streets.
2) Type B Median Break – Traditional Left-Turn Lane (Figure 1.2): This design places the left-turn lane directly adjacent to the through passing lane of the expressway and will be referred to by the term "traditional left-turn lane" throughout this report. It has a negative offset with the opposing left-turning vehicle since the right edge of the opposing left-turn lane is to the left of the left edge of the approach left-turn lane. This negative offset causes sight restrictions if the opposing left-turn lane is occupied. This design is used primarily at expressway intersections with low-volume county roads and streets, and residential /commercial driveways.

No specific guidelines for left-turn-type use on rural four-lane expressways currently exist for the state of Nebraska.

Both Type A and B designs use a 15:1 taper (longitudinal to transverse) to provide a transition for the lateral shift from the adjacent through traffic lane of the expressway to the auxiliary lane for the left-turning movement. Since the Type A left-turn bay is offset laterally, the 15:1 taper provides a tapered length of 390 feet in advance of the storage length. The Type B left-turn bay shown in the 1996 NDOR Roadway Design Manual has a 180-foot tapered length (*1*). Guidelines for the Type B left-turn bay were modified in 1999 to provide additional deceleration length for left-turn lanes at major and standard county road intersections (see Figure 1.3)(*2*).

In general, the Type A design is preferred from a safety point of view because of the improved sight distance and long deceleration lengths. However, there are increased construction and maintenance costs associated with it due to the larger paved surface area and signing components. The geometric characteristics of the offset design may also cause perceptual confusion on the part of a driver not familiar with the design. Therefore, NDOR is interested in determining which design is more cost effective. In order to determine the optimal use of each design, operational effects, safety, construction, maintenance and operating costs of both designs must be analyzed and quantified. The adequacy of the geometric configuration of

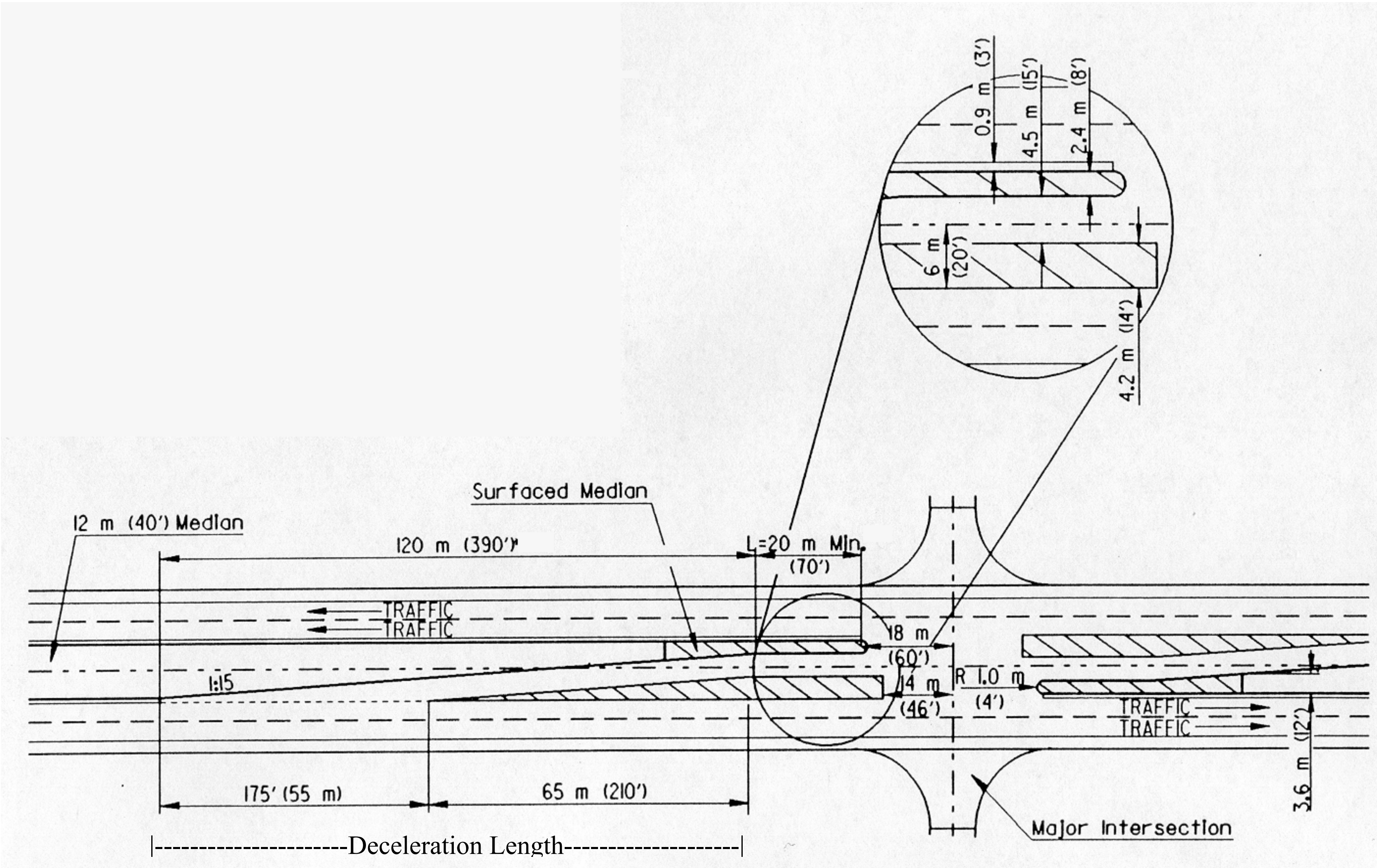
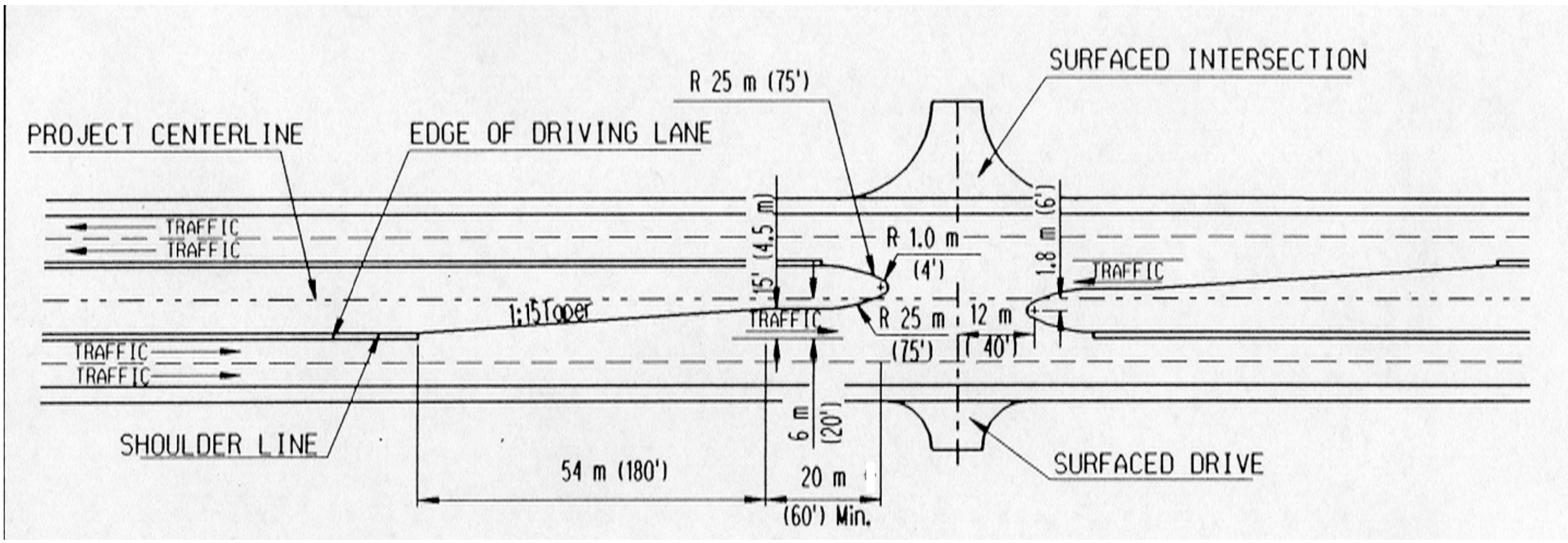


Figure 1.1 Type A Left-Turn Lane Design (*1*).



|-----Deceleration Length-----|

Figure 1.2 Type B Left-Turn Lane Design (*1*).

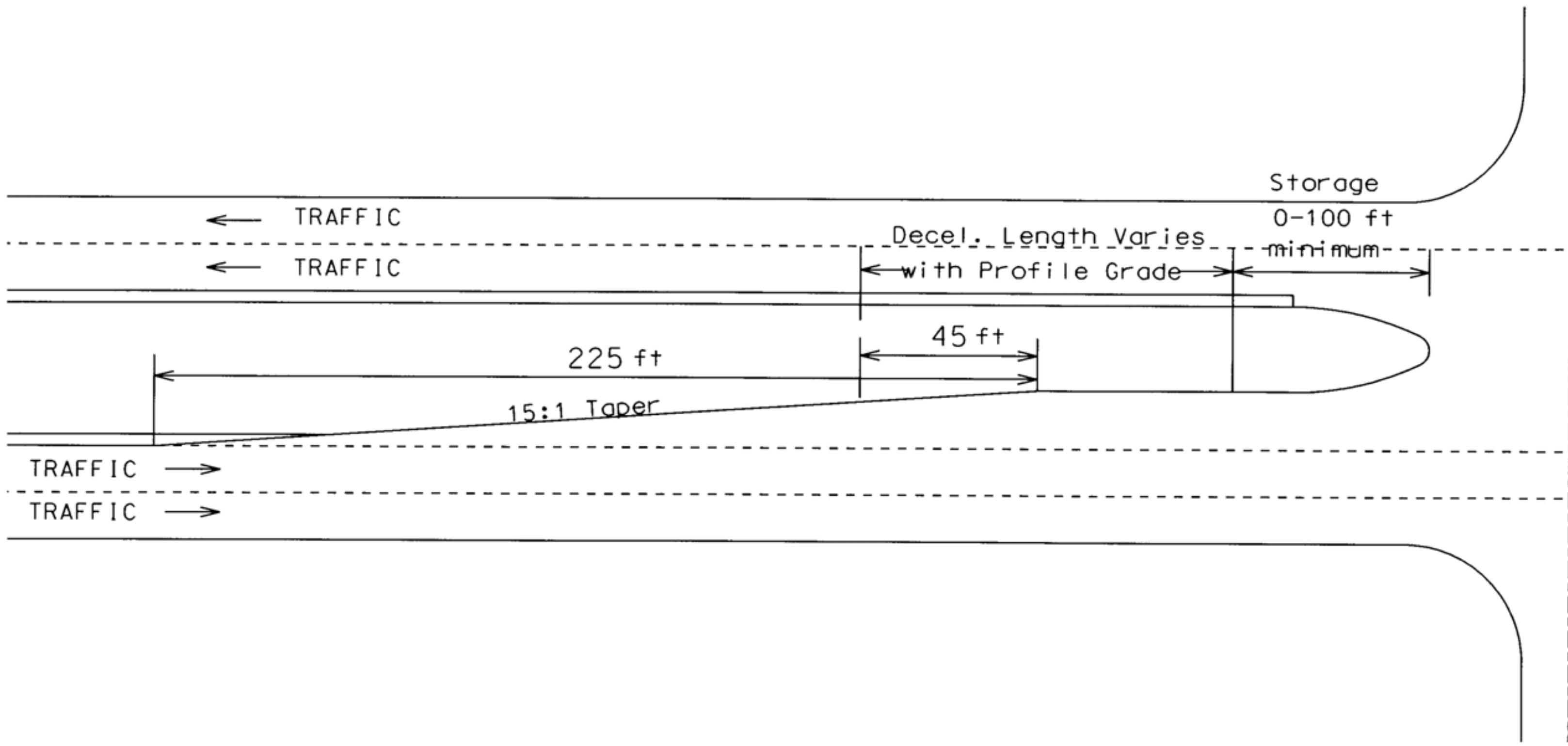


Figure 1.3 Type B Left-Turn Lane Design Modified in 1999 (2).

both Type A and Type B designs is also a point of concern. Since these left-turn lanes are adjacent to driving lanes that have been constructed to meet 70 mph design speed criteria, designs which limit the amount of deceleration of left-turn vehicles in the through lane are preferred.

Objective

The overall objective of the research project is to develop guidelines for determining which type of left-turn lane is more cost effective on rural four-lane expressways in Nebraska and to determine desirable geometric characteristics for the left-turn lane design for high-speed through traffic conditions.

The guidelines developed define the circumstances for which the construction, maintenance, and operating costs of a Type A offset left-turn lane are justified by the operational and accident cost savings it provides to road users. Recommendations are also suggested to improve the geometric features of the left-turn lanes to better accommodate the expectations and requirements of drivers making left turns from expressways.

Methodology

The development of the guidelines was based on a cost analysis of the Type A and Type B left-turn lanes. The costs used in the analysis included the construction, maintenance and operating costs of the left-turn lane along with the costs of traffic operations and accidents generated by each left-turn type.

The operational effects of left-turn lanes were determined by means of computer simulation. A microscopic traffic flow computer simulation model was used to evaluate the effects of left-turn lanes on vehicle delay and fuel consumption over a wide range of traffic conditions representative of those on rural sections of four-lane expressways in Nebraska. Multiple regression analysis of the simulation results was conducted to develop equations for vehicle delay and fuel consumption as functions of left-turn lane design type and traffic conditions.

The safety effects of the left-turn lane types were assessed in terms of the expected accidents derived from 1) speed differences between left-turning and through vehicles and 2) offset left-turn vehicles such that the critical line of sight of left-turning drivers to approaching through vehicles is uninterrupted. This researcher has developed a spreadsheet procedure to establish which left-turn lane design is most cost-effective for a particular situation. The left-turn type resulting in the lowest cost is the most desirable left-turn lane to construct.

Behaviors of drivers making left turns at high speeds were also studied and recommendations made on possible improvements to current geometric criteria based on the study findings.

Chapter 2
Literature Review

A literature review was conducted to provide a frame of reference for the research. Left-turn lanes on rural, two-way stop controlled intersections were found to have been the subject of a limited number of research studies. Even fewer involved left-turn lanes on four-lane rural expressways. The following is a review of the relevant studies on nationally accepted warrants and design guidelines for left-turn lanes on two-way stop controlled intersections, geometric guidelines from states in the Midwest region, and a study where computer simulations were used to develop related guidelines.

Purpose and Functional Elements of Left-Turn Lanes

The purpose of a left-turn lane is to allow safe deceleration adjacent to the high-speed through lanes and provide a storage area for left-turning vehicles, reducing impacts on through traffic and minimizing intersection delay. National Cooperative Highway Research Project (NCHRP) Report 279, Intersection Channelization Design Guide (*3*) provides a detailed breakdown of the functional elements of left-turn lanes. These items include the following:

- an entering tapered segment to provide a transition for the lateral shift from the adjacent through traffic lane to the auxiliary lane for the left-turn movement,
- a segment for deceleration from the through lane speed to a stop condition, and
- a segment for the storage of vehicles waiting to make a left-turn movement.

According to NCHRP 279, the bay taper design shouldn’t be so short that vehicles must decelerate abruptly in the through lane or so long that drivers mistake the left-turn lane for an additional through lane. Left-turn lanes on high-speed highways should desirably be designed to accommodate both vehicle deceleration and braking. The assumed “reasonable” driver behavior includes deceleration in gear for 3 seconds followed by comfortable braking completely within the turning lane. Where constraints exist and speeds are moderate, the initial speed of turning traffic is assumed to be 10 mph less than the design speed of the through roadway and braking is assumed to begin where two-thirds of a full-lane width is available (Figure 2.1). The storage length should be based on the rate of arrivals during an appropriate time period (*3*). The storage length should be sufficient to store the number of left-turn vehicles expected to arrive during this period without stored left-turn vehicles encroaching on the adjacent through lane more often than a specified frequency.

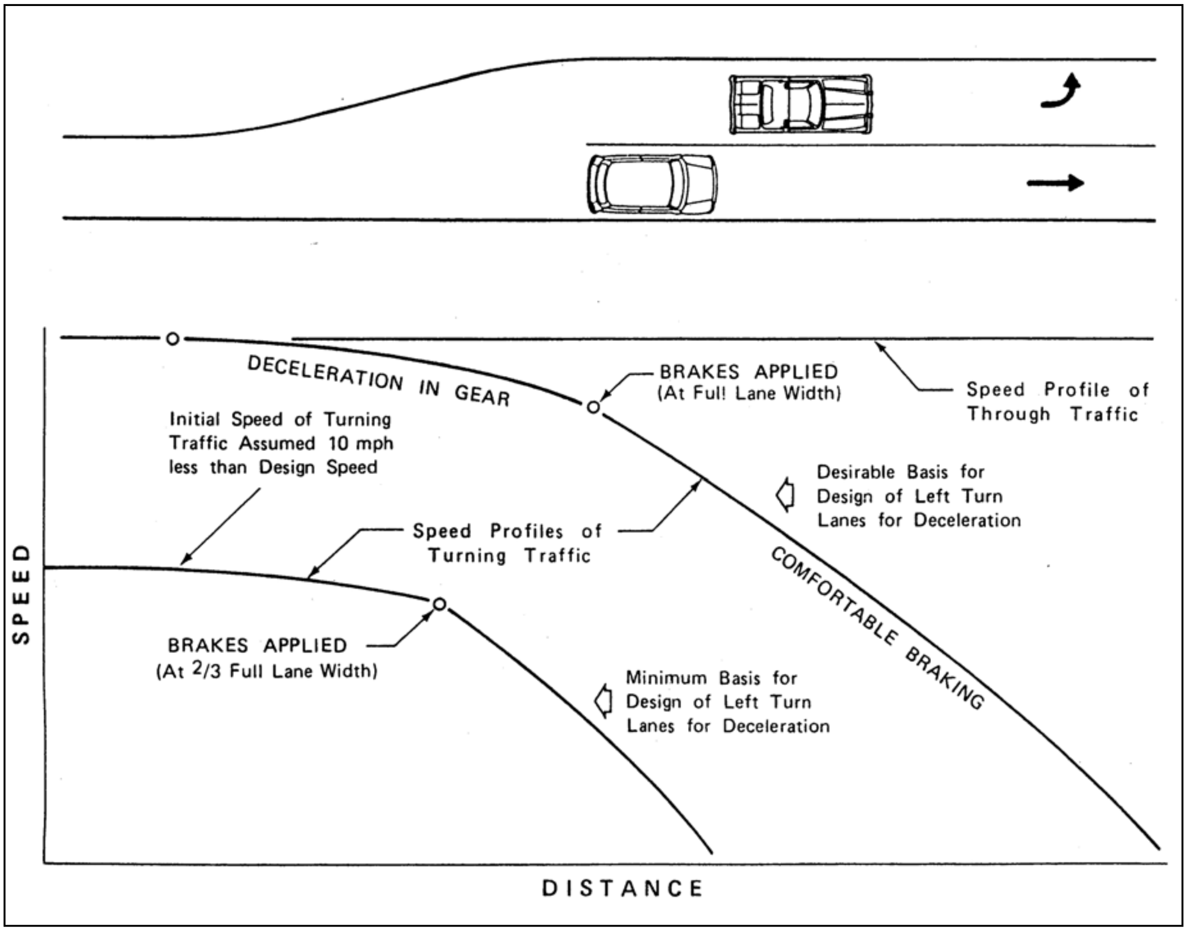


Figure 2.1 Speed Profile of Decelerating Left-Turn Vehicle (*3, Figure 4-15, p. 53)*

Deceleration Length

The American Association of State Highway and Transportation Officials (AASHTO) 2001 Policy on the Geometric Design of Highways and Streets (commonly known as the 2001 Green Book (*4*)) suggests that "provision for deceleration clear of the through-traffic lanes is a desirable objective on arterial roads and streets and should be incorporated into design, whenever practical." Table 2.1 shows values recommended as approximate total deceleration lengths needed for a comfortable deceleration from the full design speed of the highway to a stop assuming grades less than 3 percent:

Table 2.1 Desirable Auxiliary Lane Deceleration Lengths for Arterial Roads and Streets

Design Speed, mph	Desirable Deceleration Length of Auxiliary Lane, feet
30	230
40	330
45	430
50	550
55	680

The 2001 Green Book indicates that a 10-mph reduction in speed in the through lane is commonly considered acceptable on arterial roadways, but there is an indication that this only applies to urban facilities. The maximum design speed with a specific length recommendation for an arterial is 55 mph. No recommendations are given for higher design speeds, which are prevalent on rural four-lane expressways in Nebraska.

In the 2001 Green Book's discussion of single-lane free-flow exit terminals contained in Chapter 10, Grade Separations and Interchanges, Exhibit 10-73 provides guidance for minimum deceleration lengths for exit terminals with longitudinal grades of 2 percent or less. Exhibit 10-73 is shown below as Figure 2.2.

US Customary										
		Deceleration length, L (ft) for design speed of exit curve, V′ (mph)								
		Stop condition	15	20	25	30	35	40	45	50
Highway design speed, V (mph)	Speed reached, V_a (mph)	For average running speed on exit curve, V'_a (mph)								
		0	14	18	22	26	30	36	40	44
30	28	235	200	170	140	–	–	–	–	–
35	32	280	250	210	185	150	–	–	–	–
40	36	320	295	265	235	185	155	–	–	–
45	40	385	350	325	295	250	220	–	–	–
50	44	435	405	385	355	315	285	225	175	–
55	48	480	455	440	410	380	350	285	235	–
60	52	530	500	480	460	430	405	350	300	240
65	55	570	540	520	500	470	440	390	340	280
70	58	615	590	570	550	520	490	440	390	340
75	61	660	635	620	600	575	535	490	440	390

V = design speed of highway (mph)
V_a = average running speed on highway (mph)
V′ = design speed of exit curve (mph)
V'_a = average running speed on exit curve (mph)

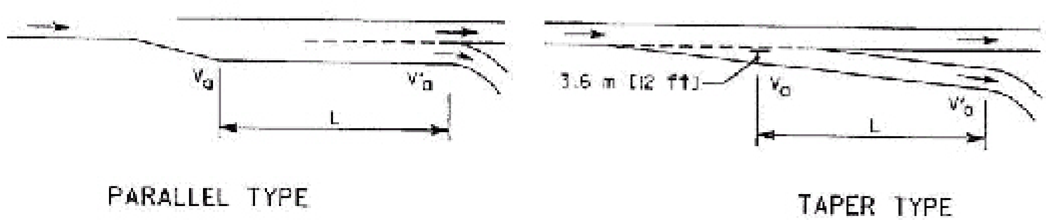


Figure 2.2 Minimum Deceleration Lengths for Exit Terminals with Flat Grades of 2 Percent or Less (*4, p. 855*)

Deceleration lane lengths for exit grades of 3 percent or greater may be accommodated by using adjustment factors given in Exhibit 10-71 of the 2001 Green Book reproduced in Figure 2.3.

US Customary					
Design speed of highway (mph)	Deceleration lanes Ratio of length on grade to length on level for design speed of turning curve (mph)[a]				
All speeds	3 to 4% upgrade 0.9				3 to 4% downgrade 1.2
All speeds	5 to 6% upgrade 0.8				5 to 6% downgrade 1.35
Design speed of highway (mph)	Acceleration lanes Ratio of length on grade to length of level for design speed of turning curve (mph)[a]				
	20	30	40	50	All speeds
		3 to 4% upgrade			3 to 4% downgrade
40	1.3	1.3	–	–	0.7
45	1.3	1.35	–	–	0.675
50	1.3	1.4	1.4	–	0.65
55	1.35	1.45	1.45	–	0.625
60	1.4	1.5	1.5	1.6	0.6
65	1.45	1.55	1.6	1.7	0.6
70	1.5	1.6	1.7	1.8	0.6
		5 to 6% upgrade			5 to 6% downgrade
40	1.5	1.5	–	–	0.6
45	1.5	1.6	–	–	0.575
50	1.5	1.7	1.9	–	0.55
55	1.6	1.8	2.05	–	0.525
60	1.7	1.9	2.2	2.5	0.5
65	1.85	2.05	2.4	2.75	0.5
70	2.0	2.2	2.6	3.0	0.5

Figure 2.3 Speed Change Lane Adjustment Factors as a Function of Grade (*4, p. 852*)

Since these recommendations are listed in the chapter discussing interchanges, they clearly apply to ramps at grade separations and are not specifically recommended for the determination of left-turn deceleration lanes.

Deceleration for a left-turning vehicle involves braking in combination with moving laterally to enter the left-turn lane. Lateral movement is commonly assumed to be 4 fps for rural conditions (*5*). If entry into the left-turn lane was initiated at 65 mph (96 ft/sec) from the through lane, the ratio of longitudinal distance to lateral movement would be 24:1. Allowing a speed differential of 10 mph in the through lane would result in an entry speed of 55 mph and a corresponding 20:1 ratio. At low deceleration rates, the driver will have shifted laterally so that a following vehicle can pass without encroaching on the adjacent driving lane before a 10 mph speed differential occurs. The speed differential will exceed 10 mph before the turning vehicle clears the through lane if the average deceleration during lateral movement exceeds approximately 3.7 fps^2 (light to reasonable braking). The left-turn lane should be designed so that a turning vehicle will develop a speed differential of 10 mph or less at the point it clears the through traffic lane (*5*).

Left-Turn Lane Taper
According to the 2001 Green Book (*4*), a taper rate between 8:1 and 15:1 (longitudinal to transverse) is commonly used for the design of left-turn lane tapers on high-speed highways. Long tapers more closely approximate the path drivers follow when entering an auxiliary lane from a high-speed through lane. However, if the left-turn lane taper is on a roadway with a horizontal curve, some drivers may be unintentionally drawn into the auxiliary lane. Some state agencies construct a "squared-off" section at full paving width and depth, and use pavement markings to delineate the taper. This bay style may have the following benefits:

1) offers improved driver commitment to the exit maneuver, and
2) contributes to driver security because of the elimination of the unused portion of long tapers.

The path a driver selects when shifting from a through lane to a full-width deceleration lane will vary depending on:

1) type of vehicle,
2) operator driving characteristics,
3) vehicle speed,
4) weather conditions, and
5) lighting conditions.

Tapers may be of the straight-line type (easier to construct) or may include short curves at either end of long tapers (the tangent section is recommended to be about a third to a half of the total length).

Storage Length
The AASHTO 2001 Green Book (*4*) provides the following guideline for the storage length of left-turn lanes. "At unsignalized intersections, the storage length, exclusive of taper, may be based on the number of turning vehicles likely to arrive in an average two-minute period within the peak hour. Space for at least two passenger cars should be provided; with over 10 percent truck traffic, provisions should be made for at least one car and one truck. The two-minute waiting time may need to be changed to some other interval that depends largely on the opportunities for completing the left-turn maneuver. These intervals, in turn, depend on the volume of opposing traffic."(*4*, *p. 718*)

Chakroborty et al (*6*) developed guidelines for determining lengths of left-turn lanes at unsignalized intersections which suggest basing lengths on turn volumes, opposing volumes, critical gap, threshold probabilities, and vehicle mix. Critical gap is the minimum time headway in the opposing flow that is required for a driver to complete a left-turn maneuver, and threshold probability defines the acceptable frequency of overflow in the left-turn lane. The study found the guidelines to be less conservative than the AASHTO guidelines in most instances.

In the 1996 NDOR Roadway Design Manual requirements for storage length follow the guideline which states that, "the storage length of a turn lane should be designed so that the number of vehicles desiring to make a turn during any interval will exceed the turn lane capacity only 5 percent of the time without reducing the safety or capacity of the approach" (*1*). A procedure developed by NDOR to obtain storage lengths calculates the average number of vehicles during an interval using the design hourly volume (30th highest hourly volume derived from an ADT volume determined for some future year) of vehicles making the turn. The average is used to find a maximum number of vehicles during the given interval based on a

Poisson distribution which exceeds 5 percent. Once the maximum number is found, it is multiplied by 25 ft (19 ft design passenger car vehicle length plus 3 ft of clear space in front of the front bumper plus 3 ft of clear space behind the rear bumper) to obtain the required storage length. Table 2.2 from the 1996 NDOR Roadway Design Manual shows a range of these storage lengths (*1*). NDOR policy agrees with the 2001 Green Book recommendation that also states that space for at least two passenger cars at a minimum (50 ft length) should be provided for storage. If the percentage of truck traffic exceeds 10 percent, storage for at least one car and one truck should be provided (75 ft of length). This recommendation conflicts with the minimum distances shown on the Type A Median Break (70 ft minimum length shown in Figure 1.1) and Type B Median Break (60 ft minimum shown in Figure 1.2) in the 1996 NDOR Roadway Design Manual (*1*). The 1999 revised Type B Median Break design matches the minimum for major intersections but not on standard county road intersections which requires no storage length.

Table 2.2 NDOR Left-Turn Lane Lengths (*1*)

Average Number of Vehicles per Interval	95% Probable Max # of Vehicles During Same Interval	Length of Turn Lane, (ft)
0.1 to 0.3	2	50
0.4 to 0.8	3	75
0.9 to 1.3	4	100
1.4 to 1.9	5	125
2.0 to 2.6	6	150
2.7 to 3.3	7	175
3.4 to 4.0	8	200
4.1 to 4.7	9	225
4.8 to 5.4	10	250
5.5 to 6.2	11	275
6.3 to 7.0	12	300
7.1 to 7.8	13	325
7.9 to 8.6	14	350
8.7 to 9.4	15	375
9.5 to 10.2	16	400
10.3 to 11.0	17	425
11.1 to 11.8	18	450
11.9 to 12.6	19	475
12.7 to 13.4	20	500
13.5 to 14.2	21	525
14.3 to 15.0	22	550

Warrants for Traditional and Offset Left-Turn Lanes

According to the 2001 Green Book, median left-turn lanes "should be provided at intersections and at other median openings where there is a high volume of left turns or where the vehicular speeds are high."(*4, p. 720*). For median widths wider than about 18 ft, "it is desirable to offset the left-turn lane."(*4, p. 727*), although McCoy et al recognized offset needs for median widths narrower than 18 ft (*7*). Offset left-turn lanes place the waiting left-turn vehicle as far to the left as practical, maximizing the offset between the opposing left-turn lanes and providing improved visibility. The 2001 Green Book does not specify values for "high" left-turn volumes or "high" vehicular speeds.

Bonneson et al (*8*) surveyed state highway departments on policies for rural expressway design practices. The survey was primarily interested in access control, traffic control, and median width, however the study did find approximately one-third of the state highway departments have successfully used offset left-turn designs at selected locations. Rarely was the design a standard, more often it was considered where wide medians exist and left-turn accident problems were encountered. The same survey found that types of corrective measures that the states apply to high-accident unsignalized at-grade intersections on rural expressways are intended to increase the likelihood of:

- attracting the driver's attention,
- attracting the driver's attention further in advance of the intersection, and
- providing more restrictive traffic regulation through the intersection.

The survey also conveyed a safety concern about driver interpretation of the small island on the right side of the offset left-turn bay since most drivers are unaccustomed to driving in offset left-turn lanes.

In NCHRP Report 375, Median Intersection Design (*9*), it was found that most states use guidelines or warrants to determine whether a left-turn lane should be provided at a particular location. Some states were found to warrant left-turn lanes when left-turn design volumes:

1) exceed 20 percent of the total directional approach design volume, or
2) exceed 100 veh/hr in the peak period.

The Illinois Department of Transportation (IDOT) uses tapered offset left-turn lanes where:

1) median widths are 40 ft or more,
2) where the current crossroad ADT (average of both approaches) is 1,500 vpd or more, and
3) where the current left-turn design hourly volume in each direction from the major road is more than 60 vph.

IDOT has found that problems with driver confusion associated with offset left-turn lanes are minimal if proper signing and pavement markings are used (i.e., advance signing and pavement arrows on the entrance to the left-turn lane)(*9*).

The Iowa Department of Transportation Roadway Design Manual (*10*) states that the use of tapered offset left-turn lanes should be considered on rural four-lane expressway intersections only if:

- there exists a likelihood that traffic signals will eventually be installed, or
- there is a significant sight distance problem caused by opposing left-turn vehicles.

The Iowa criteria also suggests a median width of 30 ft if offset left-turn lanes are used and that special attention should be given to potential median drainage issues which may be caused by the offset lane configuration.

No warrants or guidelines are currently given for the use of offset left-turn lanes on four-lane rural expressways in the roadway design manuals of states of Kansas, Missouri and South Dakota.

Geometric Guidelines for Offset Left-Turn Lanes on Four-Lane Expressways
Although the 2001 Green Book recommends the use of parallel or tapered offset left-turn lanes for median widths greater than 18 ft, it does not give specific details about appropriate geometrics. Only general figures are shown to illustrate the configurations (Figure 2.4(*4*, *p. 728*) and Figure 2.5 (*4, p. 725*)).

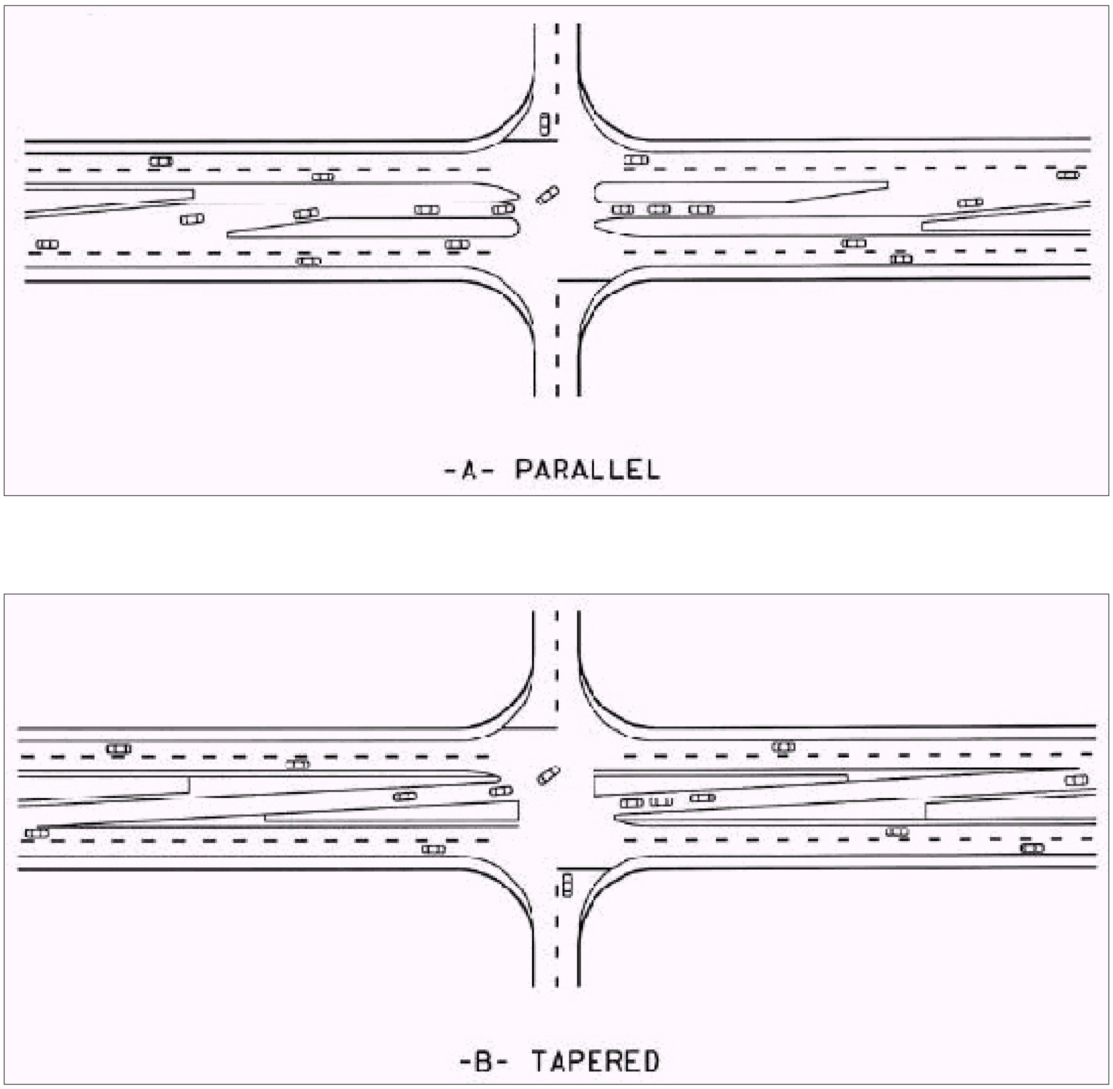


Figure 2.4 Parallel and Tapered Offset Left-Turn Lane (*4, p. 728*)

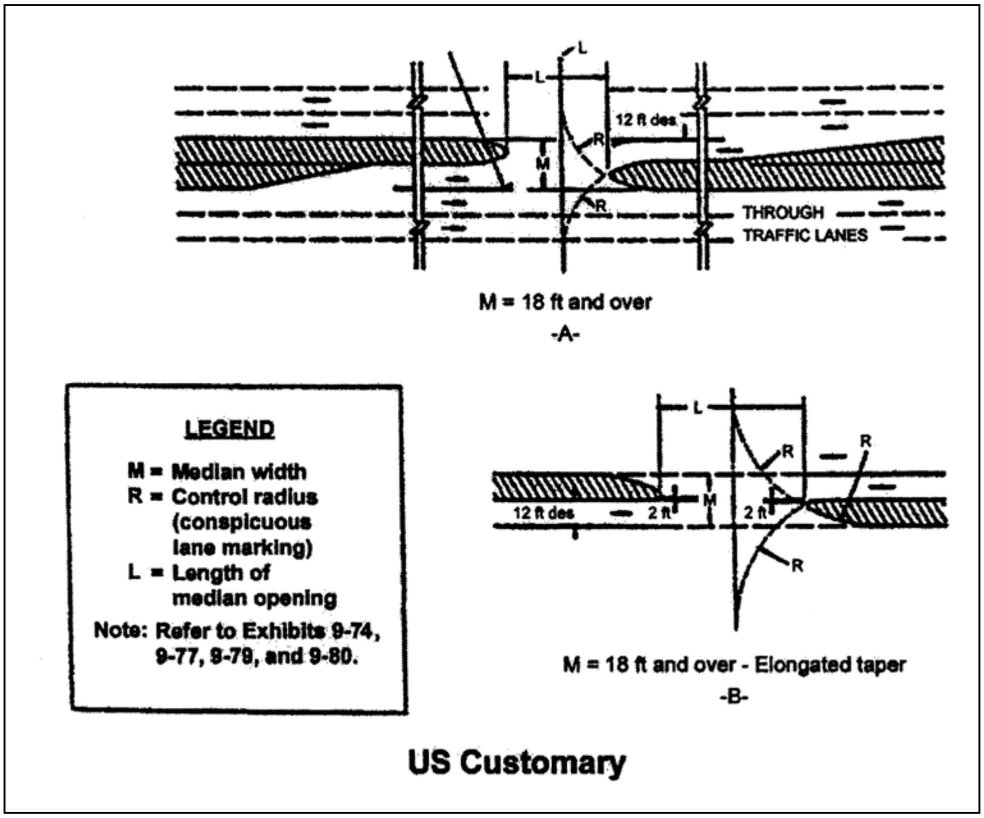


Figure 2.5 Median Left-Turn Design for Median Width in Excess of 18 ft (*4, p. 725*)

NCHRP 375 lists the following feasible allocations of available width for 40 ft medians with parallel offset left-turn lanes. These values are shown in Table 2.3.

Table 2.3 Feasible Allocations of Available Width for 40 ft Median Parallel Offset Left-Turn Lanes

Parallel Offset Left-Turn Lane Feature	NCHRP 375 Recommendation	NDOR Policy
Through-lane separator width (ft)	15	14
Left-turn lane width (ft)	12	15
Curb offset (ft)	2	Not applicable
Medial separator width (ft)	11	11
Offset to opposing left-turn lane (ft)	4	3

The geometric design of offset left-turn lanes in Nebraska is the result of an evolution of designs developed as the rural expressway system expanded throughout the state. The 1996 NDOR Roadway Design Manual shows specific geometric details for the design of a parallel offset left-turn bay on a rural four-lane expressway in Nebraska (Figure 1.1, (*1*)). The offset design (Type A Median Break, (*1*)) uses a 15:1 taper (longitudinal to transverse) to provide a transition for the lateral shift from the adjacent through traffic lane of the expressway to the auxiliary lane for the left-turn movement. The 15:1 taper provides a longitudinal length of 390 feet in advance of the storage length. A 14-ft wide flush painted island with plowable pavement markers parallels the left-turn bay lane edge to delineate the separation from the through traffic lanes. This type of design is used primarily at expressway intersections with major state highways and high-volume county roads and streets.

The state of Iowa details a two-stage taper for offset left-turn lanes in their roadway design manual (Figure 2.6, (*10*)).

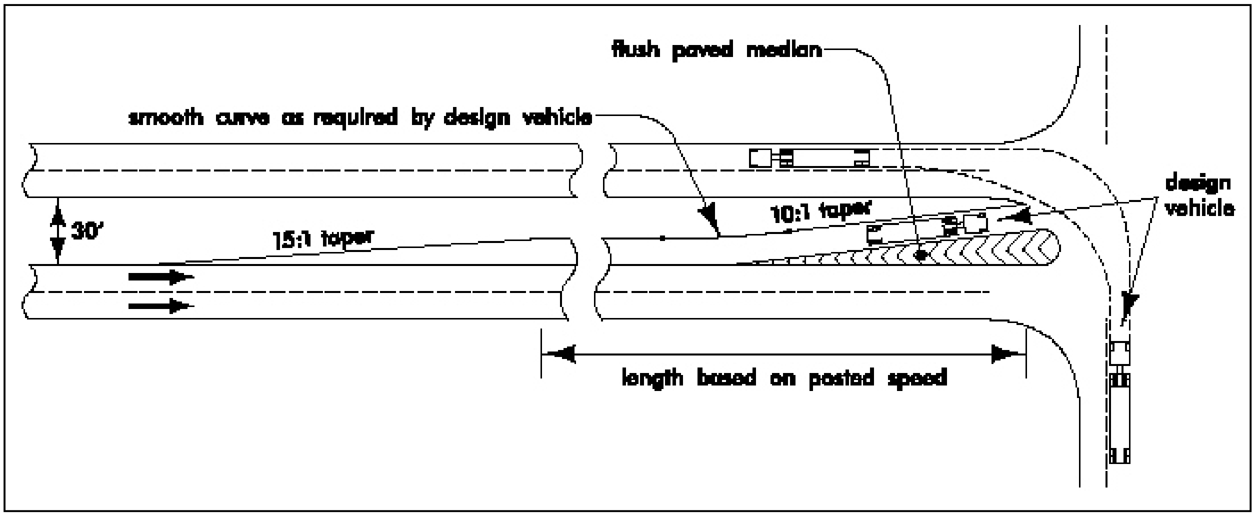


Figure 2.6 Iowa Offset Left-Turn Lane Geometrics, (*10, p. 4, Chapter 6C-5*)

Geometric Guidelines for Traditional Left-Turn Lanes on Four-Lane Expressways

NCHRP 375 lists the following feasible allocations of available width for 40 ft medians with traditional left-turn lanes. These values are shown in Table 2.4.

Table 2.4 Feasible Allocations of Available Width for 40 ft Median Traditional Left-Turn Lanes

Traditional Left-Turn Lane Feature	NCHRP 375 Recommendation	NDOR Policy
Left-turn lane width (ft)	12	15
Curb offset (ft)	2	Not applicable
Medial separator width (ft)	26	25

The NDOR may also use a traditional design for left-turn lanes along rural expressways, known as the Type B Median Break (*1*). The 1996 NDOR Roadway Design Manual has a 180-foot longitudinal distance to achieve the width of the left-turn lane which corresponds to a 15:1 taper (*1*). Guidelines for the Type B left-turn bay were modified in 1999 to provide additional deceleration length for left-turn lanes at major and standard county road intersections (Figure 1.3)(*2*)). The Type B configuration shown in the 1996 Design Manual is currently used for access to residential drives according to the 1999 revised policy (*2*).

The braking formula from the 1994 Green Book was used to determine the deceleration distance required for 1) intersections with major roads and 2) intersections with standard county roads of the 1999 revised Type B left-turn lane design. This formula is:

$$d = \frac{V^2}{254(f \pm G)} \quad \textbf{(2.1)}$$

where:

d = braking distance, m,
V = initial speed, km/h,
f = coefficient of friction between tires and roadway, and
G = percent of grade divided by 100.

The design speed of the expressway was assumed to be 110 km/h (68 mph). For major intersections, an initial speed of 85 percent of the design speed (94 km/h or 58 mph) was used based on the acceptable speed reduction of approaching mainline vehicles in Case IIIC of the 1994 Green Book's kinematic intersection stopping sight distance model. The coefficient of friction for asphalt surfacing was used in the formula since it was more conservative than the friction coefficient for concrete. Therefore, the deceleration length for each Type B left-turn lane may be designed for the deceleration along the specific longitudinal grade of the left-turn lane. A taper rate of 15:1 was still used to develop the width of the 15 ft left-turn lane (a longitudinal length of 225 ft). A 12 ft lane width is established at a longitudinal distance of 180 ft from the beginning of the taper so 45 ft of the total 225 ft may be used for deceleration. The remaining length required for deceleration to a stop is added lane length parallel to the through lanes in addition to the required storage length.

For standard county road intersections, a speed reduction of 15 km/h (10 mph) below the average running speed of 87 km/h (54 mph) was considered acceptable so a value of 72 km/h (44 mph) would be used in Equation 2.1 with the appropriate longitudinal grade to arrive at a suitable deceleration distance. As mentioned above, 45 ft of the total 225 ft taper may be used for deceleration. Table 2.5 shows a summary of geometric criteria for the 1999 revised Type B Median Break criteria.

Table 2.5 Summary of Geometric Criteria for 1999 Revised Type B Median Break

Type of Intersection	**Vehicle Speed, km/h (mph)**	**Surfacing Material For Friction**	**Grade, percent**	**Deceleration Length, m (ft)**	**Taper Rate Length, m (ft)**	**Storage Length, m (ft)**	**Total Length, m (ft)**
Major Intersections	94 (58)	Asphalt	0	129 (423)	54 (177)	15 (50)	198 (650)
Standard County Road	72 (45)	Asphalt	0	76 (250)	54 (177)	0	130 (427)
Residential Drive	61 (38)	Asphalt	0	0 (0)	54 (177)	15 (50)	69 (227)

The state of Kansas uses traditional left-turn bays on its expressways with 60-ft wide medians. Figure 2.7 shows configurations for low and high volume turning movements (*11*). A 75 m (246 ft) taper (equivalent to a 20:1 longitudinal to lateral ratio) is provided at side-road intersections when the expressway left-turn movement is less than 100 vpd with no additional provision for deceleration or storage. When the left-turn movement equals or exceeds 100 vpd, the design exhibits a taper length of 55 m (180 ft, equivalent to a 15:1 taper), a deceleration lane of 180 m (591 ft) (based on deceleration from 110 km/h (68 mph) to a stop condition), and two minutes of storage or a minimum of 15 m (49 ft) of additional length.

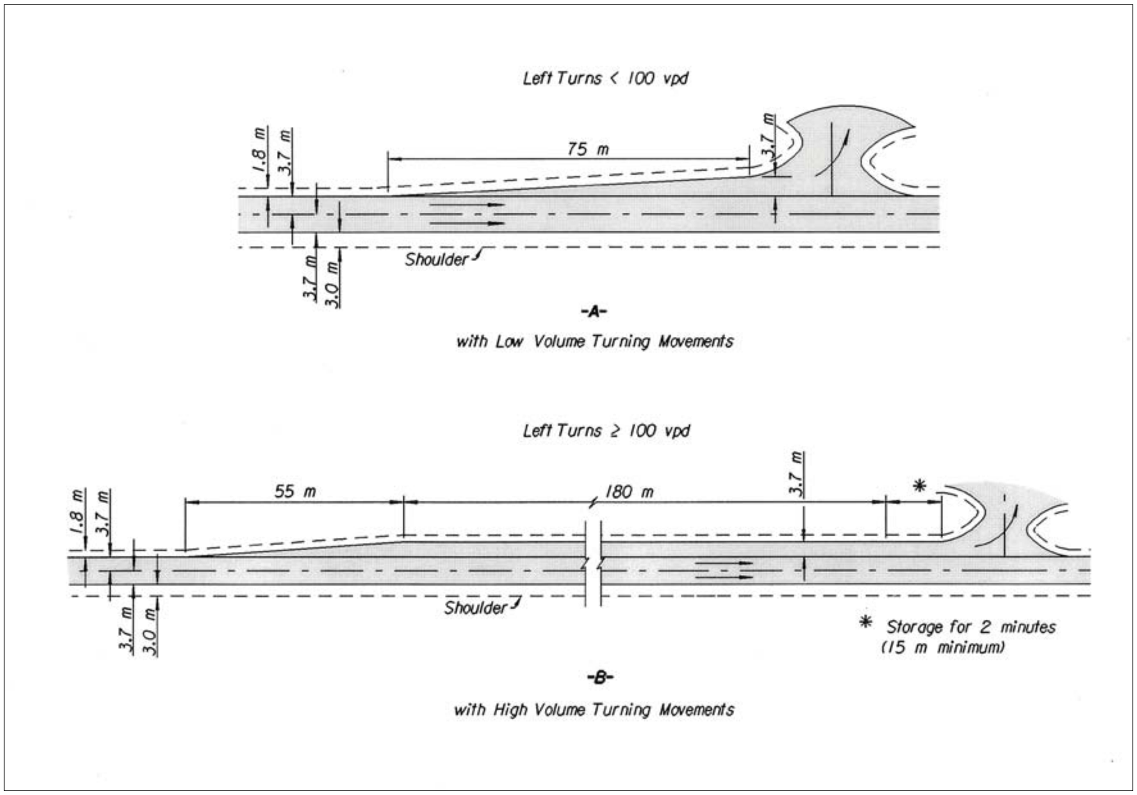


Figure 2.7 Traditional Left-Turn Lanes at Expressway Intersections in Kansas (*11*, Volume 1, pp. 7-37 – 7-39)

The Iowa Roadway Design Manual (*10*) recommends left-turn deceleration lanes with a 15:1 taper to attain a parallel lane width (Figure 2.8), then using Exhibit 10-73 in the 2001 Green Book (*4*, Figure 2.2) along with the posted mainline speed and longitudinal grade to determine the length required for a stop condition. Figure 2.9 (*10*) is used to determine the length of storage.

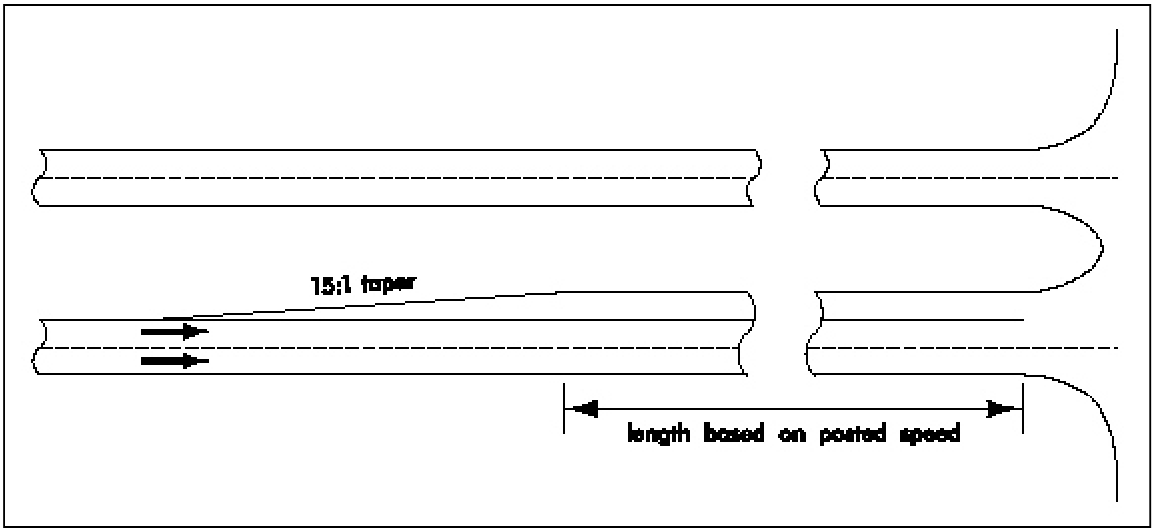


Figure 2.8 Traditional Left-Turn Lane Used in Iowa

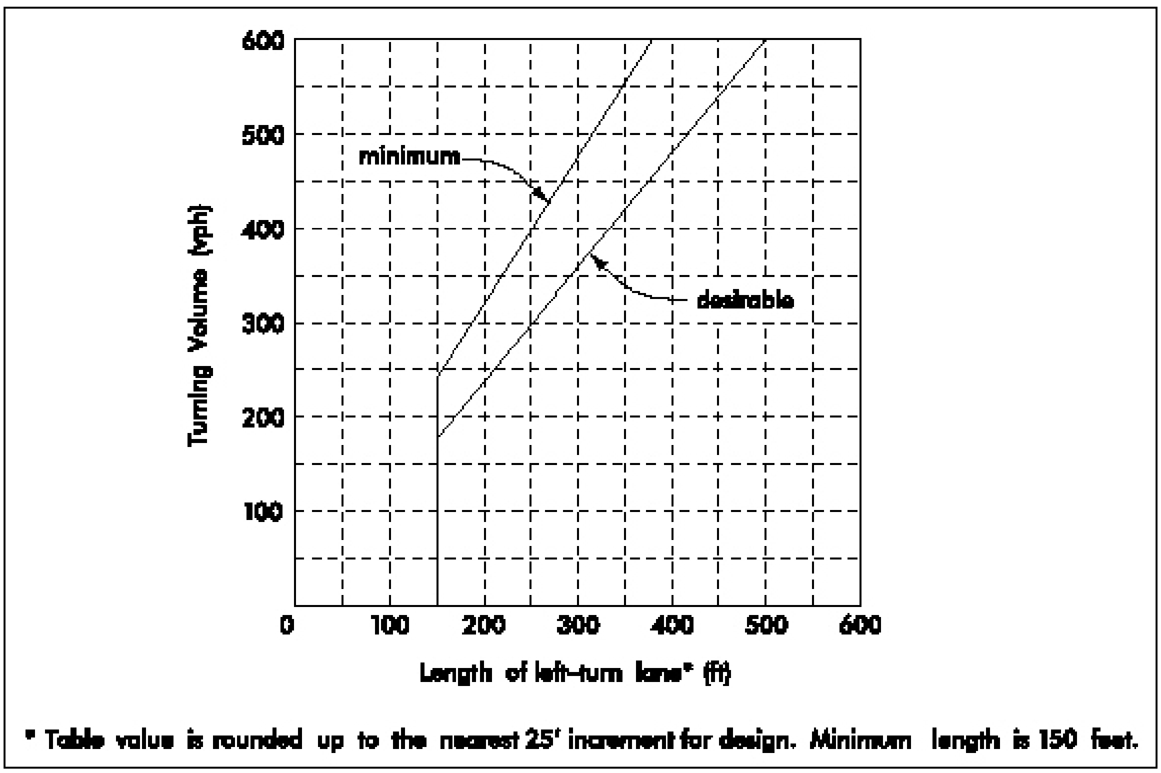


Figure 2.9 Left-Turn Storage Length Determination from Iowa ***(10, p. 4, Chapter 6A-15)***

The state of South Dakota Roadway Design Manual (*12*) modifies the recommended design for 16 to 18 ft medians in the 2001 Green Book to apply to median widths of 16 ft or greater as shown in Figure 2.10. Table 2.6 shows further modifications.

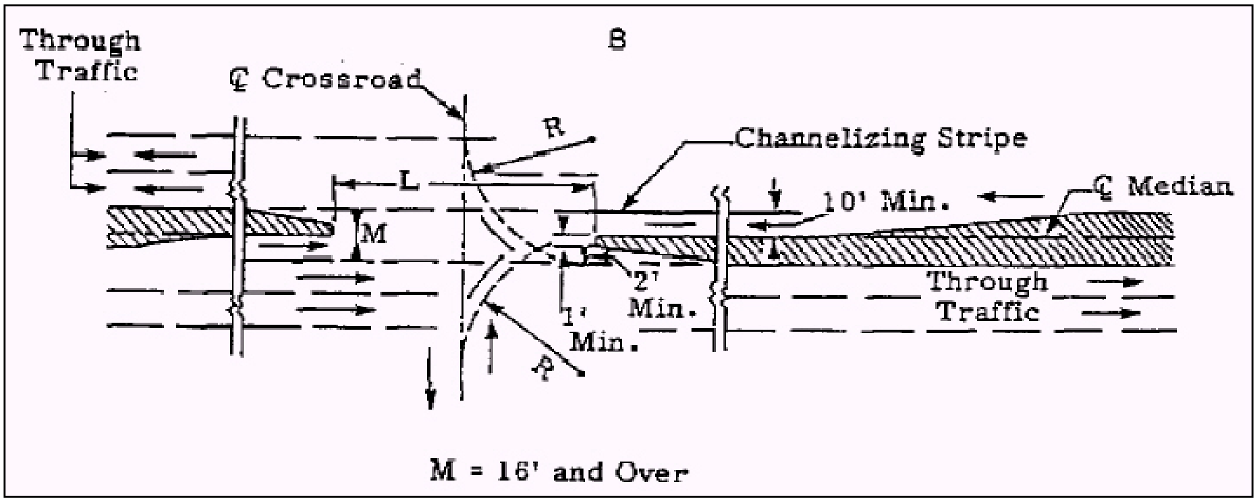


Figure 2.10 South Dakota Left-Turn Design for Median Width ≥ 16 ft (*12, p. 12-64*)

Table 2.6 Comparison of 2001 Green Book Left-Turn Lane Recommendations with those of South Dakota Roadway Design Manual

Geometric Feature	**2001 Green Book**	**South Dakota Roadway Design Manual**
Minimum Median Nose Width	4 ft	2 ft
Left-Turn Lane Width	10-12 ft	10 ft minimum
Channelizing Paint Stripe	None Specified	Specified for left-turn lane

Taper ratios used in South Dakota range from a minimum of 5:1 to a maximum of 10:1. No guidance is provided for when the minimum or maximum taper rates should be used.

The state of Missouri does not have specific guidelines for expressway left-turn lanes but does recommend consideration of left-turn lanes at intersections where the number of left-turning vehicles is 100 vph or more during the peak hour (*13*). Left-turn lanes developed from continuous medians are constructed with vertical curbs for at least the length of the left-turn storage with the vertical curb end lighted. A length of 200 ft is recommended as a desirable minimum length with adjustments made for cycle length, traffic volume and vehicle storage requirements. The use of a divisional island at an intersection to separate the left-turning traffic from same direction through traffic may be desirable, particularly where opposing traffic is separated by a wide median. No specific guidelines for the divisional island are included.

Safety Issues

In a landmark study of speed and crashes involving 10,000 drivers on 600 miles of rural highways, Solomon (*14*) found a relationship between vehicle speed and crash incidence. The relationship indicates that the accident involvement rate increases substantially when a vehicle travels much faster or slower than the average speed of traffic on a typical roadway segment. The chances of being involved in an accident are least when a vehicle is traveling near the mean speed of the traffic stream along a given roadway segment. Speed variances lower than the mean speed had a greater crash involvement rate than speeds greater than the mean speed. Figure 2.11 shows the relationship between speed differential and accident involvement rate, determined by Solomon (*14*).

West and Dunn (*15*) reported results from a study involving 216 vehicles on an Indiana state highway with speed limits in the range of 40 to 65 mph. Crashes involving turning vehicles accounted for 44 percent of all observed crashes. Data was separated into two groups, one including vehicles slowing to turn or turning across traffic and the other group excluding those vehicles. The risk of traveling much slower than average was much less pronounced in the data set without turning vehicles. The data showed little difference in crash risk for vehicles traveling within 15 mph of the mean speed of traffic. Since the primary effect of a left-turn lane is to reduce the speed differential between left-turning and through vehicles in the passing lane of a four-lane expressway, Solomon's research (*14*) will be used to quantify the safety effects of both Type A and B left-turn lane types resulting from speed reductions of left-turning vehicles. Solomon's research indicates that a reduction in the variability of speeds can be an important element in reducing crash potential (*14*).

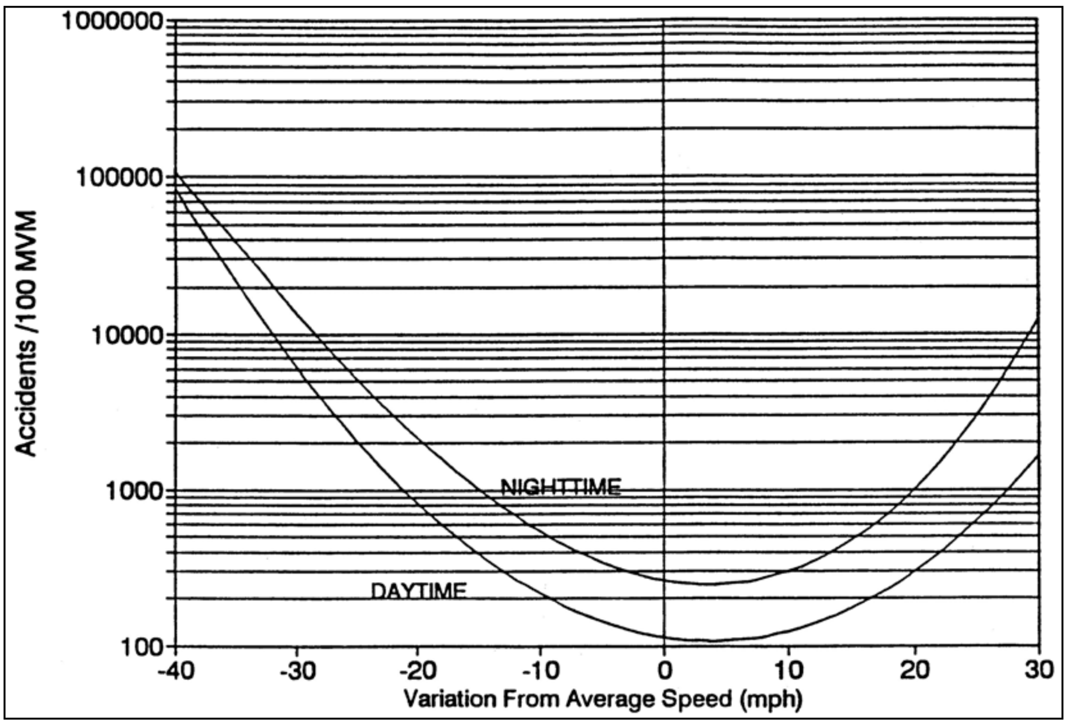


Figure 2.11 Accident Involvement Rate by Variation from Average Speed on Study Section, Day and Night, Solomon (*14*)

NCHRP Report 375 (*9*) lists several potential disadvantages of offset left-turn lanes that have been cited by highway engineers. These are:

- lack of driver familiarity with the offset left-turn lane,
- potential confusion of older drivers selecting gaps in oncoming traffic and estimating the speeds of approaching opposing vehicles (also a factor with traditional left-turn lanes),
- difficulty of snow removal and deicing activities on the separate left-turn roadway, and
- potential for wrong-way movements by opposing direction vehicles entering the left-turn roadway.

A key issue of intersection design on divided highways is whether the geometric features of the intersection lead drivers to believe that left turns in the median area should be made in front of or behind an opposing left-turning vehicle. As reported in NCHRP 375 (*9*), turn-in-front behavior predominates for intersections with medians less than 50 ft wide, and turn-behind behavior predominates for intersections with medians more than 50 ft wide.

NCHRP 375 (*9*) also suggests lighting of intersections with offset left-turn lanes should be considered whenever possible to assist left-turning drivers to recognize the proper path at night.

Previous Research Using Computer Simulation Guidelines

A study conducted on left-turn lanes in Nebraska by McCoy et al provided information and guidance pertinent to the procedure of this study (*16*). The research project developed a benefit-cost analysis procedure using accident cost savings, operational cost savings, and left-turn lane costs. However, the focus was on whether or not to provide left-turn lanes, and was not concerned with which type of design to use. The study was also based on two-lane rural highways rather than four-lane rural expressways. In regards to the operational effects portion of the study, NETSIM traffic simulations were run to compare using left-turn lanes and not using left-turn lanes over the same range of traffic conditions. These conditions included approach volumes, approach speeds, and truck percentages. Multiple regression analysis of the results from the simulation output was conducted and resulted in three regression equations for reductions in stops, delay, and fuel consumption. It was stated that "reductions in stops and delay provided by left-turn lanes to be functions of approach volume, opposing volume, left-turn volume, approach speed, and percentage of trucks." A comparable approach in the aspect of evaluating operational effects of left-turn lane types was used in this research project.

Chapter 3

STUDY DESIGN METHODOLOGY

The development of the guidelines for the determination of the optimum choice of left-turn type required the study of operations and driver behavior at rural two-way stop controlled intersections along four-lane expressways constructed with 40-foot medians in Nebraska.

Site Parameters

A rigorous attempt was made to study intersections which were constructed exactly as the 1996 NDOR Roadway Design Manual depicted them (Figures 1.1 and 1.2). As mentioned previously, the design of left-turn lanes on Nebraska expressways has evolved over an extended time period. Even the basic rural expressway median width has changed over time from 36 ft to the current width of 40 ft. The following criteria were used to locate intersections that would correctly portray general driving behavior on rural expressways in Nebraska:

- 40 ft median width,
- significant volume of left-turning traffic in daytime and nighttime,
- compliance with NDOR signing and marking standards,
- tangent horizontal alignment,
- vertical curvature design in compliance with design speed of facility,
- within a 55 or 65 mph posted speed zone (no speed transition zones), and
- study segments must be sufficiently beyond locations exhibiting platooning due to signalization in advance of the study site.

Left-turn lane designs on the rural four-lane expressways, which have stop-sign control on two-lane minor roadway intersections in Nebraska, can be grouped into four categories. These categories are distinguished by type of left-turn lane design and posted speed limit on the expressway. The four categories are:

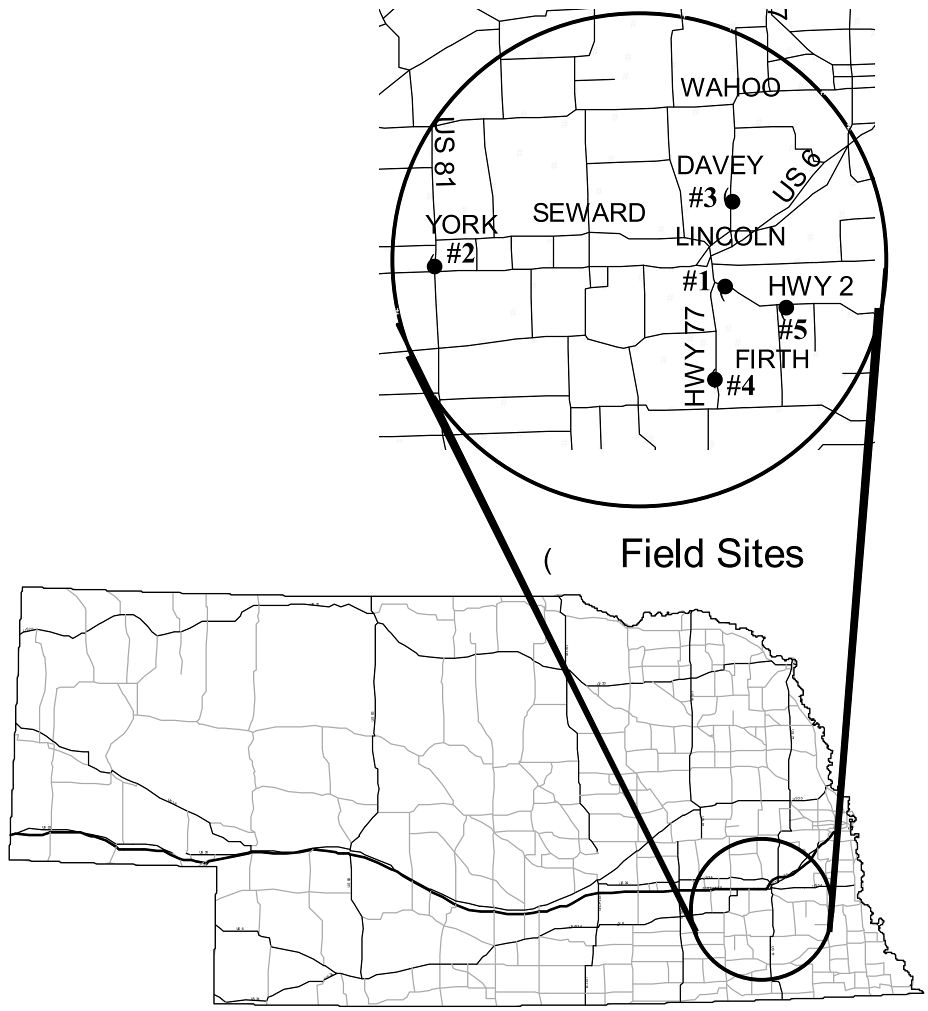
- Type A 55: "Type A" offset left-turn design on 55-mph posted speed expressway,
- Type B 55: "Type B" traditional left-turn design on 55-mph posted speed expressway,
- Type A 65: "Type A" offset left-turn design on 65-mph posted speed expressway, and
- Type B 65: "Type B" traditional left-turn design on 65-mph posted speed expressway.

Site Selection

As-built plans for rural four-lane expressways in the southeast quadrant of Nebraska were reviewed to identify potential sites that met the criteria defined above. Data was to be collected on at least one site of each of the four intersection categories. Since the literature search discovered that no relevant field data had been collected on rural offset left-turn lanes, two Type A 65 locations were chosen as study sites, one with a turf median adjacent to the left-turn storage lane and one with a surfaced median. Both sites were located on Hwy 77; one in the northbound direction, north of Lincoln, NE at the intersection with the Davey Spur (S55-E) and one south of Lincoln in the southbound direction at the intersection with the Firth Spur (S55-H). The similarity of the speed statistics at these two sites indicated need to collect data at only one representative site from each of the other three categories. A summary of study site characteristics is shown in Table 3.1. More detailed information about each site can be found in Appendix A. A map of the field site locations is shown in Figure 3.1.

Table 3.1 Summary of Study Site Characteristics, Type A and Type B Median Breaks

Study Site Number	Posted Speed Limit, mph	Left-Turn Lane Design Type/ Median Surface Type	Location	Taper Rate, Longitudinal to Lateral, ft:ft	Left-Turn Storage Length, ft	Total Length of Taper and Storage, ft	Average Daily Traffic	
							Expressway, vpd	Crossroad, vpd
1	55	A Turf	Hwy 2 - 84th St EB	7:1	275	1052	11195	Not Available
2	55	B Turf	Hwy 81 - 21st St SB	5:1	230	1000	12845	Not Available
3	65	A Surfaced	Hwy 77 - S55E NB, Davey Spur	7:1	200	848	7715	670
4	65	A Turf	Hwy 77 - S55H SB, Firth Spur	9:1	190	1100	8765	735
5	65	B Turf	Hwy 2 - 148th St EB	20:1	220	1010	11195	Not Available



#1—Highway 2 and 84th Street
#2—Highway 81 and 21st Street
#3—Highway 77 and S-55E (Davey Road)
#4—Highway 77 and S-55H (Firth Road)
#5—Highway 2 and 148th Street

Figure 3.1 Study Site Locations

Chapter 4
DATA COLLECTION METHODOLOGY

Data Collection

At each site, 14 NU-METRICS NC-97 detectors were placed on the approach of the intersection. The NU-METRICS NC-97 is a vehicle magnetic imaging traffic counter that uses the Earth's magnetic field in conjunction with a vehicle's magnetic mass to detect vehicles passing over it (*17*). The NC-97 detector records a vehicle's speed and length, tabulates volumes, and reports road surface temperature and wet/dry road surface conditions. Traffic Analyzer software is used to program and extract the data from the NC-97 detector. With this software package, the time of occurrence, speed and length of each vehicle, and one-minute pavement conditions can be recorded. Figure 4.1 shows the top view of a detector.

Figure 4.1 NC-97 Detector.

All detectors were placed in the center of the lane, except for the first two on the tapered segment of the left-turn lane. These detectors were placed in the typical vehicle path locations, which were determined by observations of left-turn vehicles. The datum point for each site was the beginning of taper location in the direction of traffic. All detectors placed upstream of the datum follow the same dimensions for both Type A and B. The detectors downstream of the datum vary according to the storage lengths of each site (Figure 4.2). Locations A and B are both half the storage length. Location C is two-thirds the distance between the beginning taper and the left-turn lane marking. Dimensions D and E are 15 ft and 6 ft, respectively. Figure 4.2 shows a general layout of the detectors and Figure 4.3 shows the placement of detectors in both through lanes of traffic.

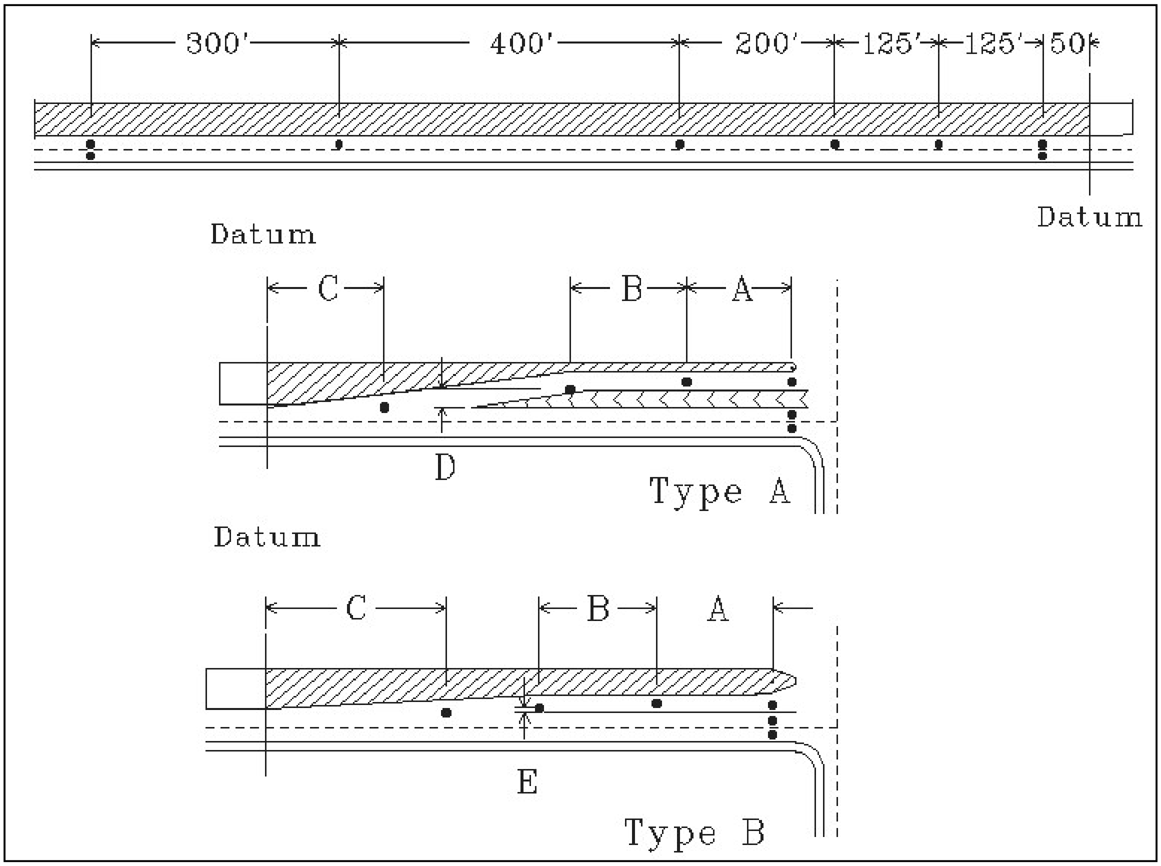


Figure 4.2 General Layout of NC-97 Detectors at Type A and B Left-Turn Lane Designs

Figure 4.3 NC-97 Detectors Installed Under Protective Covers on Expressway

Data Reduction

From these detectors, vehicle speed, length, and time of vehicle passage were collected for a period of 20 hours. The times and sample sizes of the data collection at each study site are shown in Table 4.1.

Table 4.1 Summary of Data Collected at Type A and Type B Median Breaks

Study Site Number	Posted Speed Limit, mph	Left-Turn Lane Design Type	Study Date/Time	Pvmnt. Cond.	Number of Vehicles, Cars/ Trucks	Number of Left-Turners, Cars/ Trucks	Percent of Lane Vehicles, Left/ Right
1	55	A	6-18,19-01 13:00-9:00	Dry/ Wet	3854/ 790	237/ 24	33.74/ 66.26
2	55	B	7-10, 11-01 13:00-9:00	Dry	4016/ 455	165/ 14	42.00/ 58.00
3	65	A	5-9,10-01 13:00-9:00	Dry	2469/ 292	97/ 14	21.55/ 78.45
4	65	A	6-12,13-01 13:00-9:00	Dry	3061/ 440	107/ 19	22.19/ 77.81
5	65	B	6-21, 22-01 13:00-9:00	Dry	3920/ 819	101/ 14	21.04/ 78.96

Once the data was downloaded from the detectors, a data reduction process was conducted. This process consisted of time stamping each detection event, removing speed and length outliers, and separating the data into passenger car/truck, daytime/nighttime events. Speed outliers were considered outside the range of 3 times the difference between the 25th- and 75th-percentile speeds away from the mean. Length outliers were outside the range of 5 and 81 ft. Detections during wet periods were also separated for analysis. Additional steps were conducted to determine headways between each vehicle. In order to calculate free-flow speeds, the headways of less than 5 seconds were removed from the data set (*18*).

After the collected data was reduced, the mean, 85th-, and 95th-percentile speeds were calculated for passenger car/truck, daytime/nighttime, and wet/dry pavement events. Only field data collected during clear weather and dry pavement conditions were used for model calibration purposes. Average speed differentials were also obtained for analysis by tracking each vehicle making a left-turn movement. This was accomplished by finding headway patterns along each detector in the passing and deceleration lanes. These values were used to obtain a better understanding of the free-flow speeds, deceleration patterns, and performance of through and left-turn vehicles at each site.

Data was collected at two Type A 65 sites, one with a turf median adjacent to the left-turn storage lane and one with a surfaced median. Table 4.2 shows the mean speed values for the free-flow detectors at the two sites. A t-test on the means of each detector showed no significant difference between the two locations at the 95 percent level of confidence. Therefore, only one site was used for the other three design types.

Table 4.2 Comparison of Free Flow Detector Mean Speeds.

Detector Number	Davey Site Free Flow Mean Speed [a]	Firth Site Free Flow Mean Speed [a]
1	69	66
2	66	67
3	63	63
4	N/A	67
9	61	62
13/14	67	68

[a] Detector #14 malfunctioned at Davey site, so Detector #13 is used as free flow in passing lane.

Chapter 5
CORSIM Model

The Traffic Software Integrated System (TSIS) 5.0 CORridor-microscopic SIMulation (CORSIM) software (*19*) is a microscopic traffic flow computer simulation model developed for the Federal Highway Administration. It is capable of simulating individual driver behavior in response to various traffic control parameters including the existence of left-turn lanes. The validation and application of CORSIM has been extensive making it one of the most widely used microscopic traffic simulation models. Computer simulations have proven to reduce the time and cost that would be experienced in field observations to study behaviors for varying conditions, especially at the low to moderate traffic volumes found on expressways in Nebraska.

Configuration
Left-turn lane designs were grouped into four type/posted speed categories as described in Chapter 3. Therefore, four different configurations were modeled using the TSIS 5.0 CORSIM software. The link-node diagram, as shown in Figure 5.1, is exactly the same for each model. Two, 12-ft through lanes exist in each direction, with a left-turn lane width of 15 ft and the appropriate left-turn lane configuration at the intersection. The total distance between the expressway nodes (i.e., nodes 2, 10, 1, 11, and 3) was 4000 feet in an attempt to provide sufficient distance for the arrivals at the intersection to be determined by the CORSIM traffic flow model.

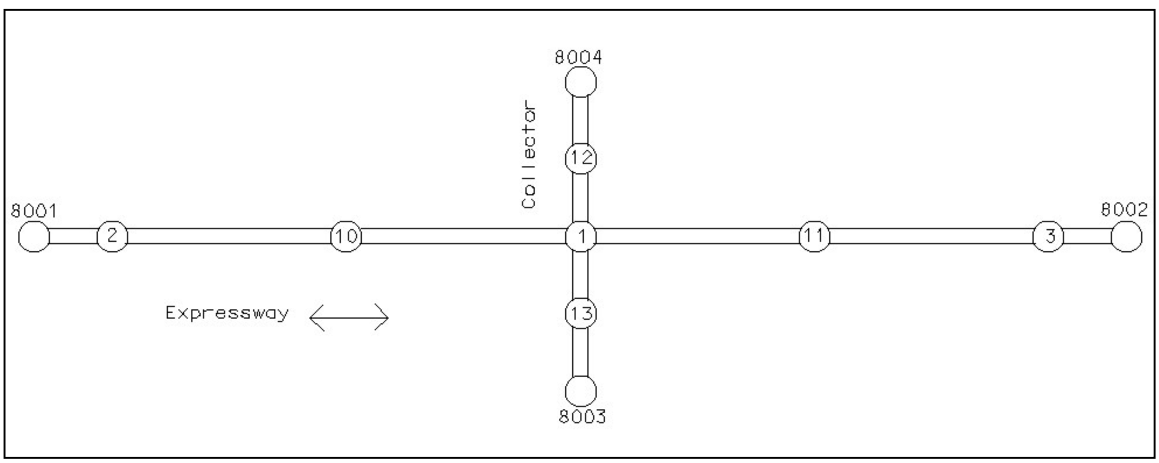


Figure 5.1. CORSIM Link-Node Diagram.

The intersecting two-lane minor roadway (Links 8004-12, 12-1, 1-13, 13-8003) was considered as a collector roadway for the left-turn vehicles off the major four-lane expressway. The differences between the models are found in the coding of the storage lengths on Links 10-1 and 11-1 and critical gap times for the left-turn vehicles.

Input
The coding of each model followed a similar process but differed in the actual values used. For example, the available gap acceptance values and storage lengths differed from Type A to Type B. Other variables included approach speeds, approach volumes, percentage of left-turns, and

percentage of trucks. These are discussed in more detail in the sensitivity analysis and experimental design.

Sensitivity Analysis

In the development of the models used in the simulations, many parameters were evaluated to determine if they had an effect on the output. This evaluation, or sensitivity analysis, examined the effects of the median coding, gap acceptance, cross traffic, and opposing traffic volumes on the model outputs. By conducting simulations that focused on the above factors, the variation in output caused by each could be determined.

Median Coding

The first factor that was analyzed was the inclusion of the 40-foot median. Due to CORSIM limitations, the software would not model any physical median width. An attempt to develop one-way pairs with an intersecting 40-foot link could not be achieved because the shortest allowable link length in CORSIM was 50 ft. Including a simulated median as combinations of closed lanes and extra turning lanes, as shown in Figures 5.2 and 5.3, made it possible to distinguish between the two left-turn designs. However, initial simulation runs proved unreliable based on the unreasonable or implausible driver decisions and vehicle paths that resulted. This led to a decision to remove the median from the model designs and use gap acceptance and left-turn storage length to account for the difference between the Type A and Type B designs.

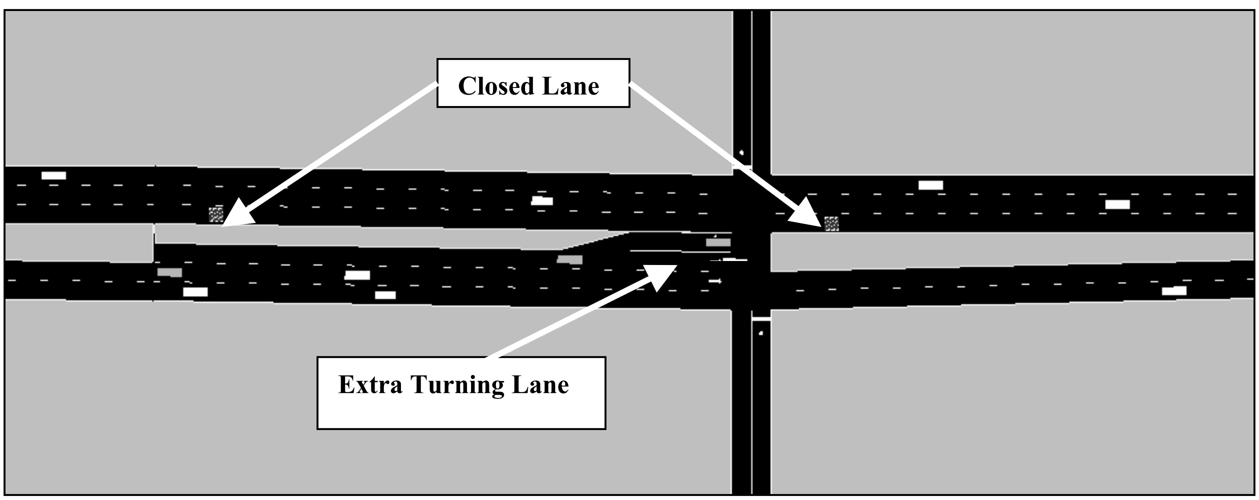


Figure 5.2. CORSIM Median Concept Design for Type A Left-Turn Lane (Rejected)

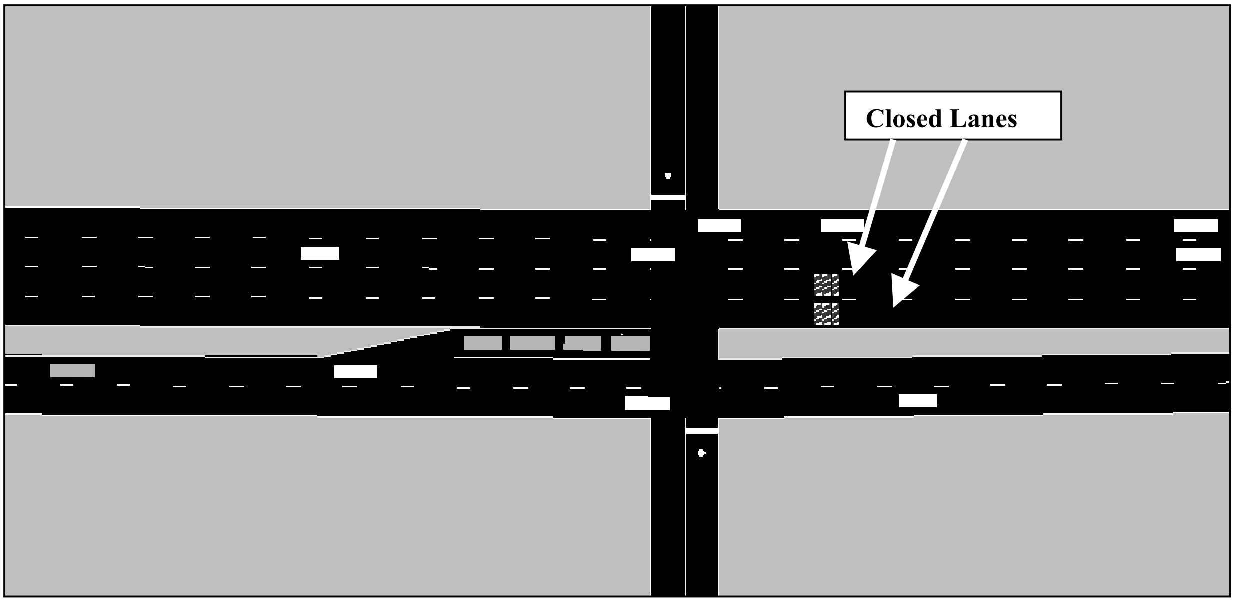


Figure 5.3. CORSIM Median Concept Design for Type B Left-Turn Lane (Rejected)

Gap Acceptance
The only coding factors that were available to distinguish the two left-turn designs were the available gap acceptance and storage length. The default values for available gap acceptance in the model ranged from 2.7 to 7.8 seconds, which were uniformly distributed among ten driver types (*19*). Each driver type had distinctive characteristics such as free-flow speed percentages, acceptable deceleration and acceleration rates, and acceptable turn gap times. The gap values were altered in order to differentiate between the Type A (offset) and Type B (traditional) designs. Due to the model's uniform distribution of driver types, a resulting mean gap acceptance was equal to 4.95 seconds. From the CORSIM documentation (*19*), the basis for these values was unclear. Therefore, 2001 AASHTO Green Book recommendations for left turns from the major road were used. The 2001 AASHTO Green Book recommends 5.5 seconds of gap acceptance time for passenger cars, plus an additional 0.5 seconds for each additional lane the turning vehicle must cross to complete the left-turn movement (*4*). As described in the model configurations, left-turn movements will have an additional lane to cross from the expressway. In addition, the Type A (offset) left-turn design has 11 ft remaining in the median width, where as the Type B (traditional) left-turn design has 25 ft remaining. Figure 5.4 shows this in detail.

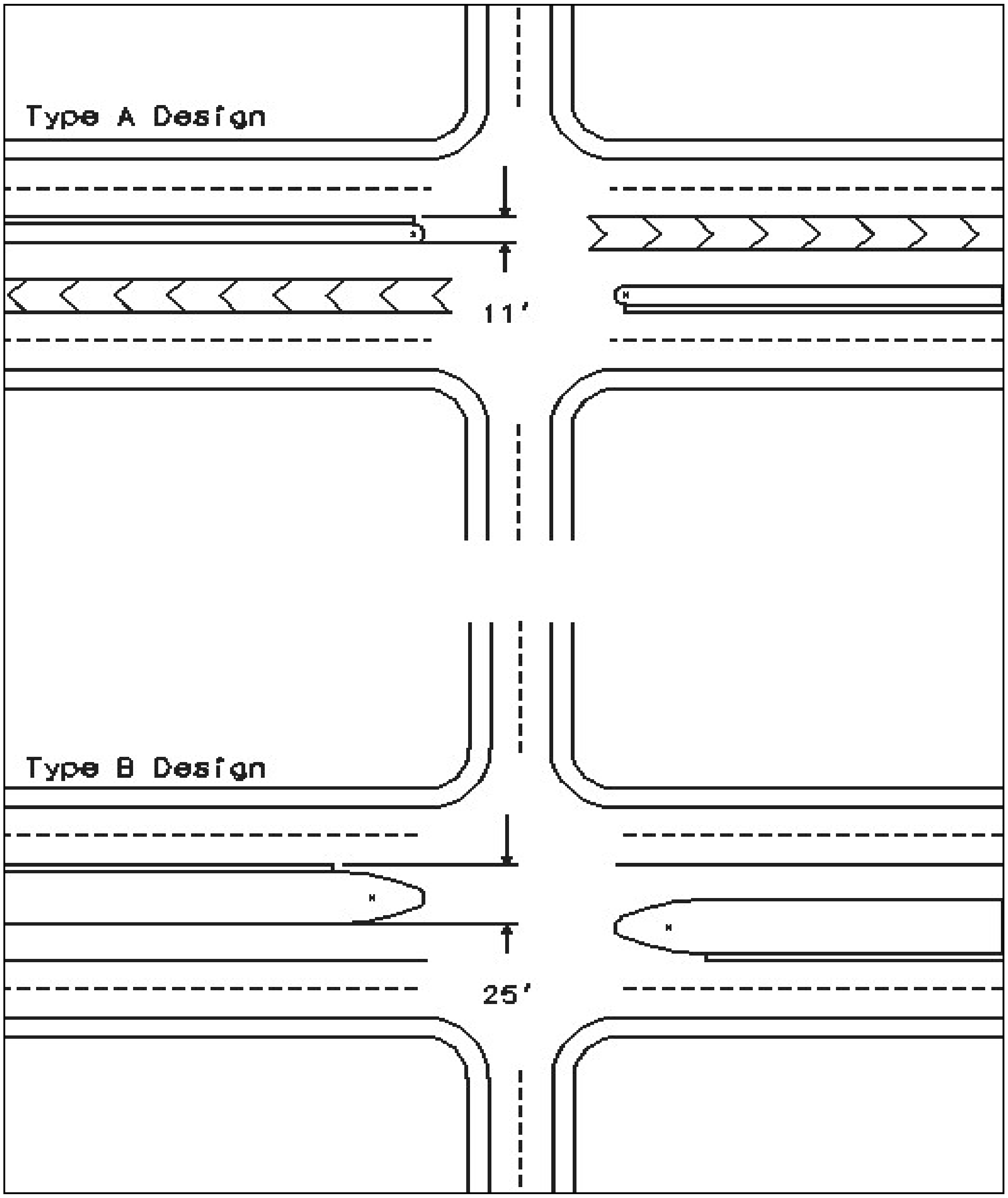


Figure 5.4. Remaining Median Width Left-Turn Movement Must Cross.

These remaining median widths in effect add one and two lanes, respectively, to the crossing maneuver. Therefore, Type A has an acceptable gap time of 6.5 seconds and Type B, a time of 7.0 seconds.

The next step was to distribute the gap acceptance times uniformly among the 10 driver types. Assuming the values were averages, the resulting range for acceptable gap times was from 3.5 to 10.2 seconds for Type A, and from 3.8 to 11.0 seconds for Type B. CORSIM only allows the values to range from 1.0 to 10.0 seconds. The results from 10 simulation test runs with the calculated ranges showed an extreme increase in the delay values and occurrences of encroachment into the through lanes. As a result, an attempt to reduce the number of driver types and in effect reduce the range of acceptable gap times lead to a decision to use only the 6.5 and 7.0 second values of gap acceptance for Type A and Type B, respectively. Examining delay to through vehicles, a comparison of the single value versus the calculated range is shown in Table 5.1.

Table 5.1. Comparisons of Through Vehicle Delay for Type B Left-Turn Lane.[a]

Case		Sample	Mean	St Dev.	P-Value	Means
2100 vph / 35% Lt	1 Value (7.0 sec)	10	12.4	0.83	0.4663	Not significantly different @ 95% confidence level
	10 Values (3.8 – 10.0 sec)		12.72	1.08		
3750 vph / 5% Lt	1 Value (7.0 sec)	10	60.27	17.25	0.2986	Not significantly different @ 95% confidence level
	10 Values (3.8 – 10.0 sec)		69.92	22.6		

[a] 250-foot Storage Length, Equal Opposing Volumes, No Trucks, No Cross Traffic.

Since the p-values in Table 5.1 are larger than 0.05, the means are not significantly different at the 95-percent confidence level. Therefore, using only one value for gap acceptance is reasonable. In choosing only one value for gap acceptance, any variance in driver aggressiveness in the turning movement is eliminated. This may cause one to believe the output to be biased. For example, those drivers that would normally accept a smaller gap size are modeled as persons that would wait for a longer gap. This results in longer delays and more vehicles queuing. However, an equal number of drivers typically wait for longer gaps and are modeled as accepting smaller gap times, which balances out any bias.

Cross Traffic

Effects of cross traffic in the simulation were examined to determine if this would have an impact on the output results and in turn increase the number of variables that would need to be included in the model regression analysis. By simulating three levels of cross traffic volume over the range of 0 to 1000 vph, no effect was observed with respect to the left-turn lane type. The Fisher's least significant difference procedure indicated no significant change in speed, percent of stops, or delay between no-cross-traffic and moderate-cross-traffic volumes at the 95 percent confidence level, as shown in Table 5.2. It is noted that the high cross traffic volume would fall under the Manual of Uniform Traffic Control Devices traffic signal warrants (*20*). The 8-Hour Vehicular Volume (Warrant 1), 4-Hour Vehicular Volume (Warrant 2), and Peak Hour (Warrant 3) are met and suggest signalization of the intersection. It was determined that due to the stop-sign control on the minor road approaches, moderate levels of cross traffic would have to wait for acceptable gaps regardless of the left-turn lane design. As a result, all the models were developed without cross traffic volumes.

Table 5.2. Cross Traffic Effects to Through Vehicles.[a]

Cross Traffic Volume	Sample	Delay	% Stops	Speed	Means
0 vph	3	12.6	11.43	49.87	Not Significantly Different
100 vph	3	11.47	12.17	50.87	
1000 vph	3	17.47	34.1	45.8	Significantly Different

[a] 250-foot Type B Left-Turn Lane, Equal Opposing Volumes, No Trucks, One Gap Value.

Opposing Traffic Volume

Opposing through traffic volumes are needed to realistically model the approach left-turn movements. The assumption of equal volumes in both directions was made to reduce the number of independent variables in the regression model to be developed. However, the need for opposing left turns is not as obvious. Intuitively, more realistic outcome in the performance measures can be expected with the inclusion of opposing left turns. By removing opposing left turns, the approaching through volumes would not be affected by vehicles attempting to make the turning movement. However, the approach left turns would be conflicted by a higher opposing through volume if the opposing left turn volume were not subtracted accordingly. Simulations were conducted with and without opposing left turns. For those runs without left turns, the percentage of left turns was subtracted from the opposing volume. For example, an opposing traffic volume of 2100 vph with 35 percent left turns resulted in 1365 vph in the opposing through direction.

Table 5.3. Comparison of With and Without Opposing Left-Turns.[a]

<table>
<tr><th colspan="2">Opposing Volume</th><th>Sample</th><th>Mean</th><th>St. Dev.</th><th>P-values</th><th>Means</th></tr>
<tr><td rowspan="4">Delay</td><td>2100</td><td rowspan="2">10</td><td>12.40</td><td>0.83</td><td rowspan="2">0.1000</td><td rowspan="2">Not significantly different</td></tr>
<tr><td>1365</td><td>13.08</td><td>0.92</td></tr>
<tr><td>3750</td><td rowspan="2">10</td><td>60.27</td><td>17.25</td><td rowspan="2">0.0653</td><td rowspan="2">Not significantly different</td></tr>
<tr><td>3563</td><td>47.72</td><td>10.55</td></tr>
<tr><td rowspan="4">% Stops</td><td>2100</td><td rowspan="2">10</td><td>12.32</td><td>1.59</td><td rowspan="2">0.5942</td><td rowspan="2">Not significantly different</td></tr>
<tr><td>1365</td><td>12.72</td><td>1.70</td></tr>
<tr><td>3750</td><td rowspan="2">10</td><td>18.87</td><td>3.95</td><td rowspan="2">0.4454</td><td rowspan="2">Not significantly different</td></tr>
<tr><td>3563</td><td>17.78</td><td>1.98</td></tr>
<tr><td rowspan="4">Speed</td><td>2100</td><td rowspan="2">10</td><td>50.04</td><td>0.76</td><td rowspan="2">0.1187</td><td rowspan="2">Not significantly different</td></tr>
<tr><td>1365</td><td>49.45</td><td>0.84</td></tr>
<tr><td>3750</td><td rowspan="2">10</td><td>27.35</td><td>4.74</td><td rowspan="2">0.0916</td><td rowspan="2">Not significantly different</td></tr>
<tr><td>3563</td><td>30.75</td><td>3.73</td></tr>
</table>

[a] 250-foot Type B Left-Turn Lane, No Trucks, One Gap Value.

Table 5.3 compares volumes of 2100 vph with 35 percent left turns and 3750 vph with 5 percent left turns. There is no significant difference in delay, percent stops, and speed at the 95 percent confidence level when opposing left-turns are included. It was decided to include the opposing left-turn volumes with the same percentage as in the approach direction. This reduced the number of independent variables that needed to be considered in the subsequent regression analysis of the simulation output.

Calibration

Calibration of the computer simulation model to accurately represent field conditions is important for producing output that is similar to what can be expected in the real world. Therefore, model parameters were inspected to see if they matched those observed in the field. Calibration checks for entry headways and speed parameters are discussed below.

Entry Headways

Vehicle entry headways were the first simulation parameter examined in the calibration process. The entrance headway is the time between vehicles entering the system at Nodes 8001 and 8002. The CORSIM model allows uniform, normal, and four versions of Erlang distributions of entry headways. Each site was evaluated to determine what type of distribution the headways might represent. Headways at each site were categorized into the 20 hours represented in the data collection time period, and the four peak-hours were evaluated. These hours were between 15:00pm and 19:00pm. In a uniform distribution, headways greater than 24 seconds result in volumes lower than 150 vph, which the research did not examine. Therefore, all headways greater than 24 seconds were not included in the evaluation. This was also done in order to focus on the smaller headways. It was found that the observed headways follow a composite distribution which cannot be modeled in CORSIM. A statistical analysis on the overall fit of each distribution suggested the Erlang with $\alpha = 1$ had the closet fit, however the Erlang distribution with $\alpha = 4$ was chosen to represent the observed headways because it provided the closest visual fit to the headways of less than 12 seconds. Comparative runs in the simulation model of the two Erlang distributions showed no significant difference between the values as shown in Table 5.4.

Table 5.4 Comparison of Observed Headway Distributions with Erlang with $\alpha = 4$ and Erlang with $\alpha = 1$.[a]

Volume (vph)	Delay (veh/sec)			% Stops			Speed (mph)		
	E4	E1	p-value	E4	E1	p-value	E4	E1	p-value
1000	2.4	2.5	0.4227	0	0	0.4538	61.3	61.1	0.3892
2000	6.3	6.2		0.8	0.7		56.3	56.4	
3000	95.5	106.3		26.1	28.0		19.8	18.4	

[a] 65 mph, 10% Left-Turns, 10% Trucks.

The 15:00 comparison for Hwy 77 and Firth Road can be seen in Figure 5.5.

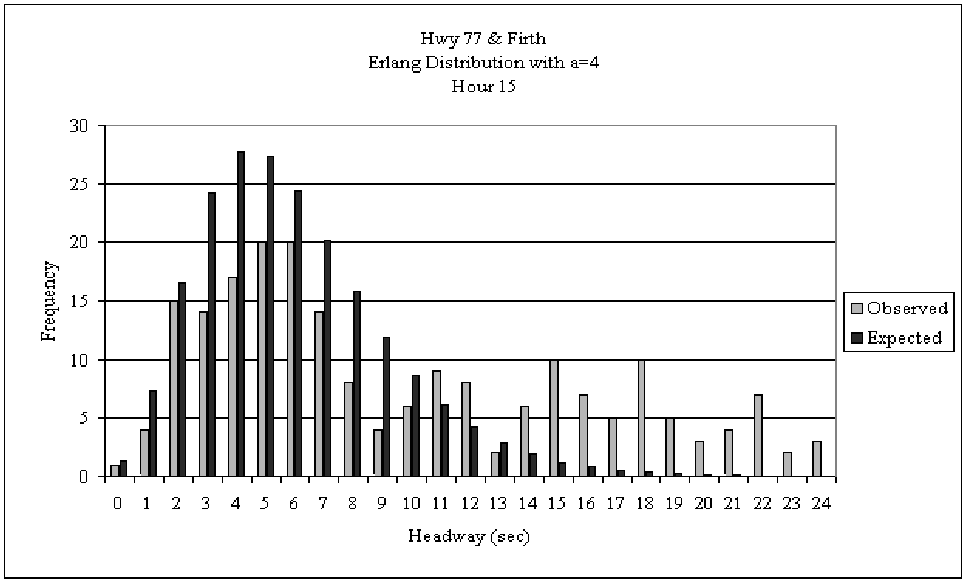


Figure 5.5. Headway Distribution Comparison for Hwy 77 & Firth Road

Speed Parameters

In order to determine if the developed models accurately represented the field data, comparisons of speeds were made. Of the five field sites where data was collected, only two sites could be used for model calibration because of the CORSIM model's limitation of a maximum of 65 mph for free-flow speed. The three unused sites had mean free-flow speeds greater than 65 mph. Simulation models were developed that represented the same characteristics, such as free-flow speed, volume, percent left-turns, and percent trucks as the field sites. The values for each of the characteristics used in the models are shown in Table 5.5.

Table 5.5 Site Characteristics Used in Model Calibration.

Study Site Number	1				2			
Location	Hwy 2 & 84th St.				Hwy 81 & S 21st St.			
Left-Turn Type	A				B			
Storage Length	275 ft				230 ft			
Posted Speed	55 mph				55 mph			
% Trucks	17.0				10.2			
Time Mean Speed (mph)	58.71				56.27			
Volume Time Periods	15:00-16:00	16:00-17:00	17:00-18:00	18:00-19:00	15:00-16:00	16:00-17:00	17:00-18:00	18:00-19:00
Hour Volumes (vph)	462	547	578	380	434	497	520	363
% Left-Turns	5	6	5	6	3	3	5	5

The simulations examined a four-hour period with varying volumes and left-turn percentages of those found in the field. The four-hour period was from 15:00 to 19:00. Simulation presence detectors were also placed in the CORSIM model at the same relative locations as the NU-METRICS NC-97 detectors in order to collect speed data. From the simulation runs, the speeds were collected from the detectors located 50 feet upstream of the datum point (the point where the taper begins). The speeds were weighted to find an average speed for the approach. These values were compared using a paired t-test on the weighted means of the field data to determine if there were significant differences between the CORSIM model and actual field data. The mean speeds at each hour and the p-values for each site are shown in Table 5.6. The tests show that no significant differences existed between the speeds of the model and field observations at the 95 percent level of confidence. This suggests that the model accurately represented approach speeds found at the study sites.

Table 5.6 Validation of Simulation Model

	Hwy 2 & 84th St			Hwy 81 & S 21st St.		
Hour	**Field Mean Speeds**	**Model Mean Speeds**	**p-value**	**Field Mean Speeds**	**Model Mean Speeds**	**p-value**
15	59.0	59.0	0.8717	56.8	57.1	0.9535
16	59.2	58.8		57.6	56.7	
17	57.3	58.6		56.6	56.2	
18	59.1	58.5		56.5	57.4	

Chapter 6
Experimental Design

In order to develop the operational effects equations that could be used for various traffic conditions, it was necessary to define independent variables and their value ranges to be used in the simulation. These ranges had to have realistic values for the expressways in Nebraska. Based on the results of the sensitivity analysis, the number of variables was minimized. In addition, the values of the variables included were limited to focus on the primary differences between the two left-turn lane design types found in Nebraska. The following is a description of each independent variable and the range in values used in the simulation.

Gap Acceptance

Being able to distinguish between the two left-turn designs was a key feature in the choice of variables. As mentioned previously (in Chapter 5), the gap acceptance is a major component in the difference. The longer a critical gap, the longer a vehicle remains queued. If conditions are right, spillback (encroachment and queuing on the through lanes) can occur which results in safety concerns for approaching vehicles in the passing lane of the expressway. All simulation runs that were conducted for Type A used a critical gap acceptance value of 6.5 seconds, and Type B simulations used 7.0 seconds as described in the sensitivity analysis included in Chapter 5.

Storage Length

In the process of distinguishing between the two left-turn designs, it was also decided that storage length for the left turns played an important role. Under Nebraska design guidelines for Type A and B designs, left-turn lanes must provide a minimum of 70 feet for storage or exceed storage lane capacity only 5 percent of the time during any interval (*1*). The observed field site storage lengths ranged from 190 feet to 275 feet. Therefore, the minimum storage length of 70 feet and a representative maximum storage length of 250 feet were used in the simulation models.

In order to further distinguish the designs, the models were adjusted for additional taper storage. An offset left-turn lane can be considered to have more storage length then a traditional design if the tapered portions of the designs are included for storage. Understanding that both designs use a 15:1 taper and multiplying the ratio by the distance from the edge of median to the edge of the through lane minus 9 ft for buffering from through traffic, an equivalent storage length can be determined. Under this assumption, a Type A offset left-turn lane (which has an effective lane width of 29 ft) can add an additional 300 ft of storage. A Type B traditional left-turn lane portrayed in the 1996 NDOR Roadway Design Manual (*1*) which has an effective lane width of 15 ft results in an additional 90 ft of storage. Figure 6.1 shows the additional storage lengths of each design. As a result, Type A designs had equivalent storage lengths of 370 ft (70 + 300 ft) and 550 ft (250 + 300 ft), and Type B designs had equivalent storage lengths of 160 ft (70 + 90 ft), and 340 ft (250 + 90 ft).

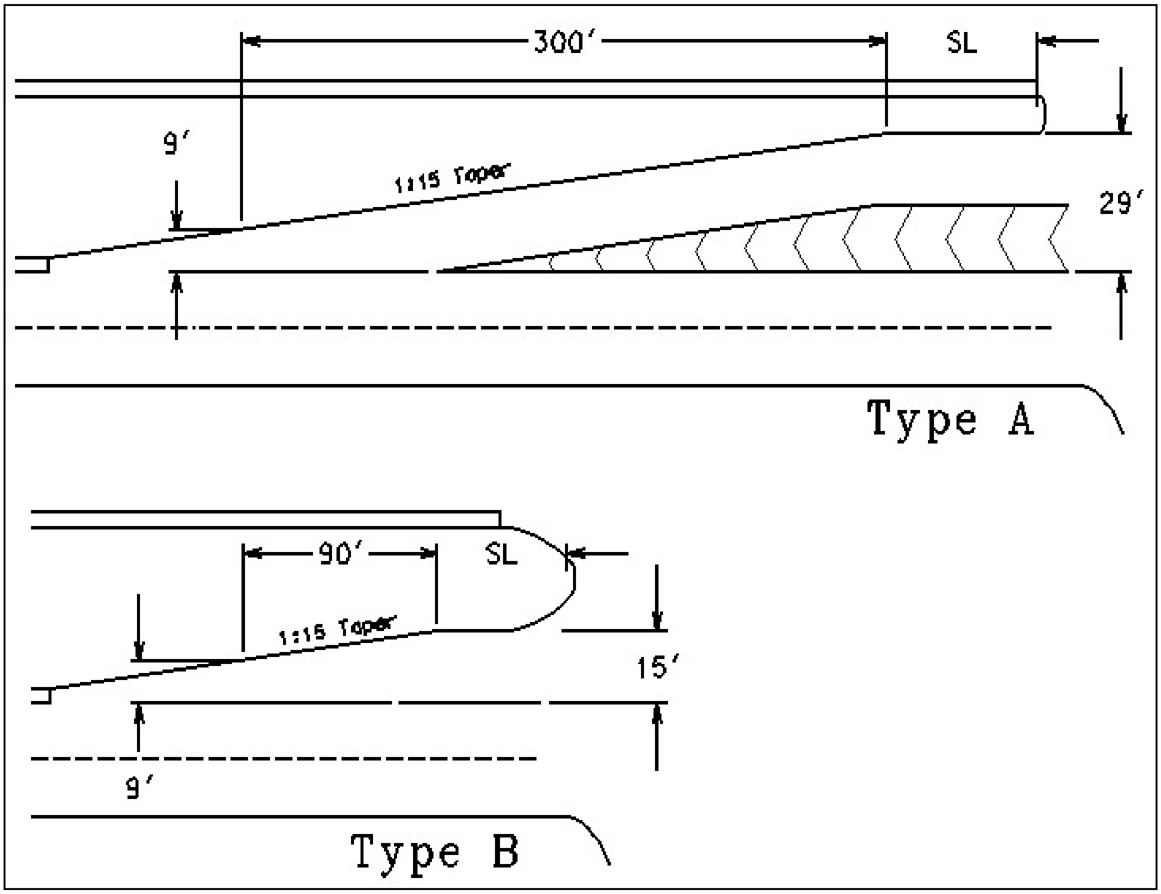


Figure 6.1. Equivalent Storage Lengths.

Approach Speed

Another variable that was used in the simulations was the approach speed. This factor affects the overall delay and fuel consumption of vehicles that travel the roadway. For example, higher approach speeds will require greater deceleration for a left-turn movement. This, in turn, can affect the following vehicles in a similar manner. Initially, the models were developed based on posted speeds. However, in the actual running of the simulations, an average time-mean speed was calculated from the field data for each type to better represent the actual field conditions. As a result, the models with a 55 mph posted speed limit were simulated using a free-flow time-mean speed of 57 mph. The time-mean speed for the 65-mph posted speed limit models was 68 mph. However, limitations in the simulation software prevented input of free-flow speeds higher than 65 mph. As a result, the approach speed used for these two models was 65 mph.

Approach Volume

Approach volume was another variable that was used in the simulations. The volume plays an important role in the operational effects. Higher volumes increase delay and reduce speeds. Values for this variable were based on current and projected annual average daily traffic (AADT) volumes for rural Nebraska expressways found in the 2000 State Highway Inventory Report

(*21*). Higher AADT volumes were also included to create equations that could be used in other regions with higher AADT values than in the state of Nebraska. The high and low extremes were then divided into five categories for simulation. On rural roads with average fluctuation in traffic flow, the 30th highest hourly volume of the year is typically about 15 percent of the AADT (*4*). It was assumed that, for rural conditions, a 50/50 split in directional flow is reasonable since split information in rural areas is generally not readily available to the designer when making choices of which left-turn lane type to select for construction. Therefore, the directional design hourly volume (DDHV) was calculated as 15 percent of half the value of the AADT. Table 6.1 shows the AADTs and the DDHVs used in the simulations.

Table 6.2. Volumes Used in Simulations

Present/Projected AADT (vpd)	DHV (vph)[a]	DDHV (vph)
2000	300	150
14000	2100	1050
26000	3900	1950
38000	5700	2850
50000	7500	3750

[a] 15% of AADT

Left-Turn Percentage

An obvious variable that had to be used in the simulation process was the percent of left turns. Varying the number of left turns would give a better understanding of how each design is affected with regard to operations. Changing the left-turn percentage shows how each design deals with varying levels of demand. It also helps determine the effects on through vehicles and left-turn vehicles still in the passing lane when storage space is fully utilized. In determining what range of percentages to use, it was decided that 5, 20, and 35 percent would be used. With a maximum of 35 percent, over a third of the total approach volume will be turning left at the intersection, which may seem extreme in certain instances, but could be a possibility. No guidelines or warrants for left-turn percentages at unsignalized intersections were found in the literature. However, Harmelink used a range of 5 to 40 percent left-turns in the approach volume in the nomographs for determining volume warrants for provision of left-turn lanes (*22*).

Truck Percentage

Another input variable in the simulation was the percent of trucks in the traffic stream. This too can influence many of the operational effects since trucks have lower vehicle performance measures than passenger cars. Many roadways vary in the percentage of truck traffic. Based on heavy truck percentages found in the 2000 Continuous Traffic Count Data and Traffic Characteristics on Nebraska Streets and Highways (*23*), values of 0, 15, and 30 percent were used in the simulations. No truck percentages higher than 30 percent were found on Nebraska expressways, so this seemed a logical maximum.

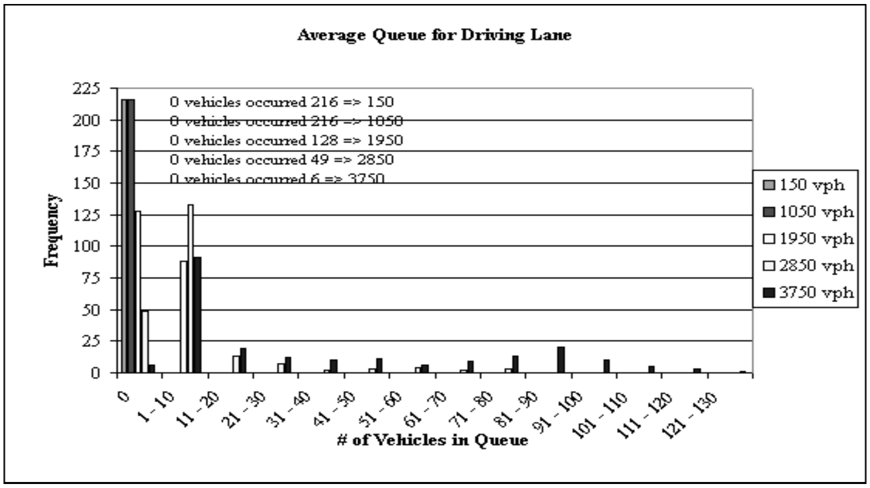
Simulation Runs

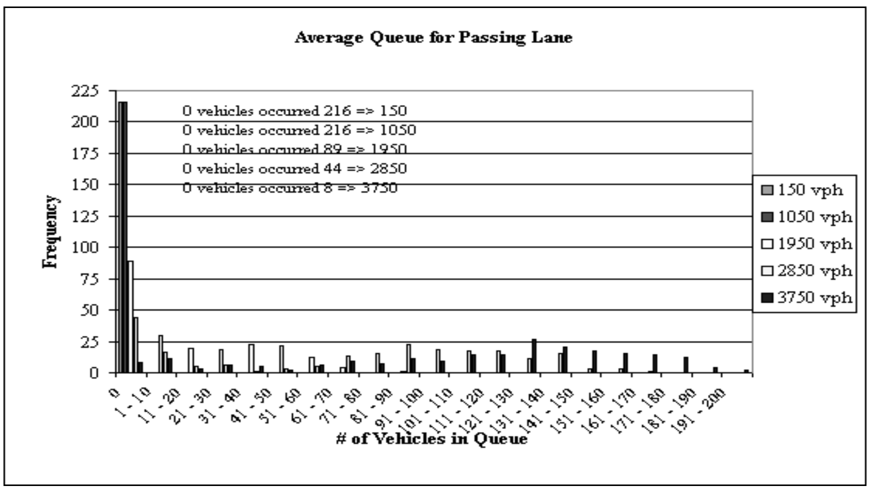
The six independent variables previously described resulted in 360 different combinations. Three 15-minute runs of each combination were simulated resulting in 1080 total simulation runs. Since two-thirds of the simulation output was used for model formulation and the remaining one-third for model validation, three runs of each combination allowed for easy splitting of the simulation data for each combination. Each of the three runs differed in the combination of driver type and vehicle type, meaning particular driver aggressiveness features were matched with a different vehicle for each run. Therefore, each simulation had different output which helps in finding an average value by increasing the sample size. Due to limited time and resources, it was not possible to perform a greater number of simulations.

Chapter 7
Regression Analysis

Performance Measures
Several performance measures were available for analysis from the output of the simulation models. These included average and maximum queue in each lane, delay, percent stops and average speed for through and left-turn movements, discharge volumes for each lane, and fuel consumption in miles per gallon. Performance measures such as delay and fuel consumption can be assigned a monetary value, and therefore were selected to develop operational effects equations. A monetary value can also be assigned to stops but stops were taken into account by delay so they were not included as a separate measure of operational performance. Relevant measures of these include: through vehicle delay in seconds per vehicle and total fuel consumption on the approach in gallons of fuel per 15 minutes. These measures were collected from the approach link 10-1 (Figure 5.1) of the intersection in the simulation model and used as dependent variables in the regression analysis.

Model Formulation
Independent Variables
The average queue for each lane on the approach was plotted as a function of each independent variable in an attempt to determine which independent variables were most meaningful. Average queue was examined because it in effect covered delay and stops and therefore did not require examining each dependent variable relationship. Histograms of frequency versus the number of vehicles in the queue for each variable were developed. Figure 7.1 shows a sample of the histograms used in the analysis.





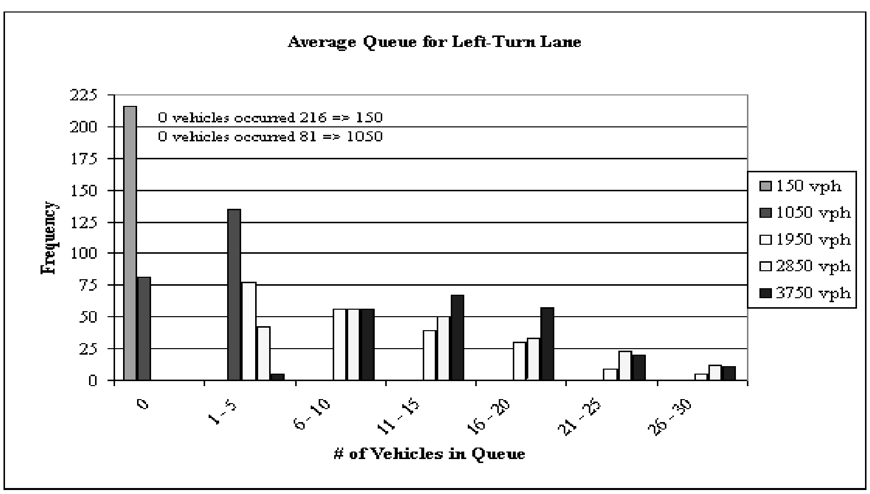


Figure 7.1 Histograms of Average Queue in Each Lane of Approach

From a visual inspection of the histograms, it was determined that each of the following variables seemed to have a noticeable effect on the average queue:

- gap acceptance times,
- equivalent storage lengths,
- approach speeds,
- approach volumes,
- percentage of left-turns, and
- percentage of trucks.

Therefore, these variables were selected for inclusion among the independent variables considered in the analysis.

Two-thirds of the simulations were used for model formulation and the remaining one-third for model validation. This provided 720 samples for the regression analysis, and 360 samples for validation purposes. The simulations were divided up based on the random seed numbers used in the modeling process. For each combination, the same two random seed numbers were selected for formulation, and the remaining seed number was selected for validation. The output data was then entered into Statgraphics Plus 5.0 (*24*), a statistical analysis software program capable of performing multiple regression analyses.

In some of the simulations, left-turn volumes exceeded the capacity of the left-turn lanes, and therefore, spillback occurred. Initial model formulation examined whether including simulations where capacity was exceeded limited the models' accuracy. Linear models of delay to the through vehicles were created using a forward stepwise multiple regression analysis. After evaluating the adjusted R^2 values of the model with all simulations versus the model with only simulations not exceeding left-turn lane capacity, it was determined that all simulations should be included in the model formulation. Not only were the adjusted R^2 values higher, but also all distinguishing variables between the Type A and B left-turn lanes were dropped from the final model when simulations exceeding capacity were eliminated.

Comparisons of adjusted R^2 values of models with and without constants were conducted. Through this process, a decision to formulate the final models without constants proved better in explaining the variance in the data and in describing situations with no traffic. For example, if all the independent variables would be zero, there would be no experienced delay. The decision to use forward stepwise regression was also made in order to simplify the model by removing the independent variables that are not significant at the 95 percent confidence level.

Models

Initial models were developed using linear relationships. However, the fit of the delay model resulted in an adjusted R^2 value of 71, so natural log relationships were also considered for all models in an attempt to increase the adjusted R^2 values. Correlation matrices were calculated to identify multi-colinearity, or linear relationships amongst the independent variables. A pair of variables with a correlation coefficient above 0.5 or below –0.5 were considered correlated. A positive correlation indicates that the variables vary in the same direction while a negative correlation indicates that the variables vary in the opposite direction. Procedures of eliminating variables that were correlated resulted in more intuitively correct models. Examination of the coefficients for each variable were performed in order to better understand the effect each variable had on the final outcome.

Many of the initial models developed showed correlations between gap acceptance and other independent variables. An attempt to develop equations for each type of left-turn design further increased the number of correlated independent variables. As a result, individual design-type models were not considered and gap acceptance was eliminated from the final models. Several models also had correlations between equivalent storage length and approach volume or approach speed. In an attempt to improve the fit of the models, approach volume and approach speed were replaced by equivalent density in the analysis.

Many of the exponential relationships still contained correlations between equivalent length and truck percentages, so a polynomial examination of the independent variables was investigated. The polynomial model resulted in an acceptable adjusted R^2 value with no correlation amongst the variables. However, negative delay values for certain traffic conditions existed and lead to the decision to use the developed natural log equations.

The independent variables considered throughout the analysis process were:

- gap acceptance times, seconds,
- equivalent storage length, ft,
- approach volume, vph,
- approach speed, mph,
- left-turns in approach volume, percent,
- trucks in approach volume, percent,
- density (approach volume/approach speed) on approach link, vehicles per mile,
- approach volume squared, vph^2,
- approach speed squared, mph^2, and
- equivalent storage length squared, ft^2.

The results of the regression analysis developed the following exponential relationships for delay and fuel consumption.

$$DL = e^{[\,-0.00161\,\mathbf{EL} + 0.0170\,\mathbf{LT} + 0.0740\,\mathbf{D}\,]} \quad \mathbf{(7.1)}$$

where,

DL = delay experienced on approach link, seconds per through vehicle,
EL = equivalent storage length, ft,
LT = percentage of left-turns in approach volume, percent, and
D = density on approach link, vehicles per mile.

$$FC = e^{[\,-0.000546\,\mathbf{EL} + 0.00550\,\mathbf{LT} + 0.0120\,\mathbf{T} + 0.000918\,\mathbf{V}\,]} \quad \mathbf{(7.2)}$$

where,

FC = fuel consumption on approach link, gallons per 15 minutes,
EL = equivalent storage length, ft,
LT = percentage of left-turns in volume, percent,
T = percentage of trucks in volume, percent, and
V = approach volume, vph.

The final model adjusted R^2 values, variables, coefficients, and correlation values of both models can be found in Table 7.1. The correlation between length and density in the delay equation suggests that when storage length increases the density decreases. There is no logical justification for this relationship and since the correlation is borderline, it was ignored.

Table 7.1. Summary of Operational Effects Models.[a]

Measure	Adjusted R^2	Variables		Correlations Between	
		Natural Log (Forward Stepwise)	Coefficients		
Delay Thru (sec/veh)	95.8	Equiv. Length	-0.00161	Length - Density	-0.5573
		% LT	0.0170		
		Density	0.0740		
Ave. Gallons / 15 min	95.2	Equiv. Length	-0.000546	No correlation between variables	
		% LT	0.00550		
		% Trucks	0.0120		
		Volume	0.000918		

[a] No constants in models, all simulations w/o gap acceptance in analysis

Model Validation

The validation process used the remaining one-third of data collected from the simulation runs to compare the model output to the simulation output. The independent variables were input into the equations and the mean absolute errors were determined. Paired t-tests comparing the difference between the means of the mean absolute errors were performed to determine the models' appropriateness. The mean absolute errors and standard deviations for the developed model and the validation calculations along with the p-values of the hypothesis tests are shown in Table 7.2. All tests at the 95 percent confidence level resulted in not rejecting the null hypothesis that the difference in the means was equal to zero. As a result, the models were considered to be valid for the purpose of this study.

Table 7.2. Summary of Validation Results.

Measure	Mean Absolute Error		Standard Deviation		P-Value
	Model	Validation	Model	Validation	
Delay Thru (sec/veh)	14.73	13.64	32.73	28.38	0.5719
Ave. Gallons / 15 min	4.08	3.96	4.63	4.31	0.6736

Results

The operational effects equations were developed through the process of computer simulation and multiple regression analysis. The delay to the through vehicles uses an exponential relationship (Equation 7.1) to describe 95.8 percent of the variation due to storage length, left-turn percentage, and density. The model suggests a proportional increase in delay as density and left-turn percentage increase, and an inverse proportionality as storage length increases. This is a logical result for the delay to through vehicles. As more storage length is provided, left-turn vehicles use the storage lane to decelerate rather than using the through lanes, which in turn reduces delay to the through vehicles.

In the fuel consumption equation (Equation 7.2), an exponential relationship describes 95.2 percent of the variation on the number of gallons consumed during a 15-minute period, due to effect of storage length, left-turn and truck percentages, and volume. All factors increase consumption proportionally except for storage length, which has an inverse effect. Fuel consumption is based on acceleration and deceleration, so less fuel will be consumed if less variation in speed occurs. By providing longer storage lengths, turning vehicles can vacate the through lanes at higher speeds and thus create smaller variations in speed changes for following vehicles.

Limitations

The operational effects equations are limited to the range of values that were used to develop them. It cannot be determined how reliable the outcome of the equations would be for values of volume, speed, left-turn percentage, truck percentage, and storage lengths outside the ranges considered in this analysis.

Chapter 8
SAFETY EFFECTS

Background

To adequately compare Type A and B left-turn lanes, safety aspects of both configurations must be examined. Four predominant safety issues are addressed in this research study when comparing the Type A offset left-turn lane to the Type B traditional left-turn lane. These issues are:

1) speed differentials between high-speed through vehicles and decelerating left-turn vehicles in the passing lane,
2) sight obstructions caused by opposing left-turn vehicles,
3) reasonable deceleration rates, and
4) driver expectations concerning turning paths through the median intersection area.

It is understood that left-turn lanes improve the safety of traffic operations by removing the deceleration of left-turning vehicles from the through traffic lanes thereby reducing the potential for rear-end collisions. It would seem logical that since the Type A offset left-turn lane has a greater offset than the Type B traditional design shown in the 1996 NDOR Roadway Design Manual (*1*), the length for deceleration is effectively increased by a distance of 210 ft at a 15:1 taper rate. Thus, the opportunity for left-turning vehicles to enter the Type A left-turn bay at a higher speed than the Type B exists.

The Type A offset design also shifts the left-turn storage lane laterally in the median closer to the opposing traffic, which reduces the sight distance restriction caused by opposing left-turning vehicles.

Reasonable deceleration rates are expected to be associated with safe left-turn operations. The Type A design, with its longer deceleration length, would be expected to provide for more reasonable deceleration rates than the Type B design.

Another possible safety benefit of the Type A design is associated with driver expectations concerning turning paths through the median intersection. The Type A offset design may result in a decreased possibility of conflict between opposing left-turn movements within the median intersection turning area. As mentioned in NCHRP 375 (*9*), turn-in-front behavior predominates for intersections with medians less than 50 ft wide, and turn-behind behavior predominates for intersections with medians more than 50 ft wide. Since Nebraska uses a 40-ft median width which is near the 50-ft behavior-change value of wide paved intersection area, the Type B design may encourage some drivers to turn behind rather than turn in front of an opposing left-turn vehicle, thus creating more conflict between the left-turn movements.

This chapter describes an evaluation of the operations of the left-turn lane designs with respect to these four safety issues.

Speed Differential

As stated in Chapter 2, the research done by Solomon (*14*) determined a relationship between speed differential and accident involvement rate. The accident involvement rate is a measure of the driver's chance of being involved in a crash at any particular driving speed. Regardless of the mean speed on the highway segment, the greater the driver's variation from this mean speed, the greater the opportunity for crash involvement, according to Solomon (*14*) as shown in Figure 8.1. This relationship was used to estimate the number of accidents caused by the difference in speed between through vehicles in the passing lane and those vehicles decelerating to enter a left-turn lane on rural expressways in Nebraska.

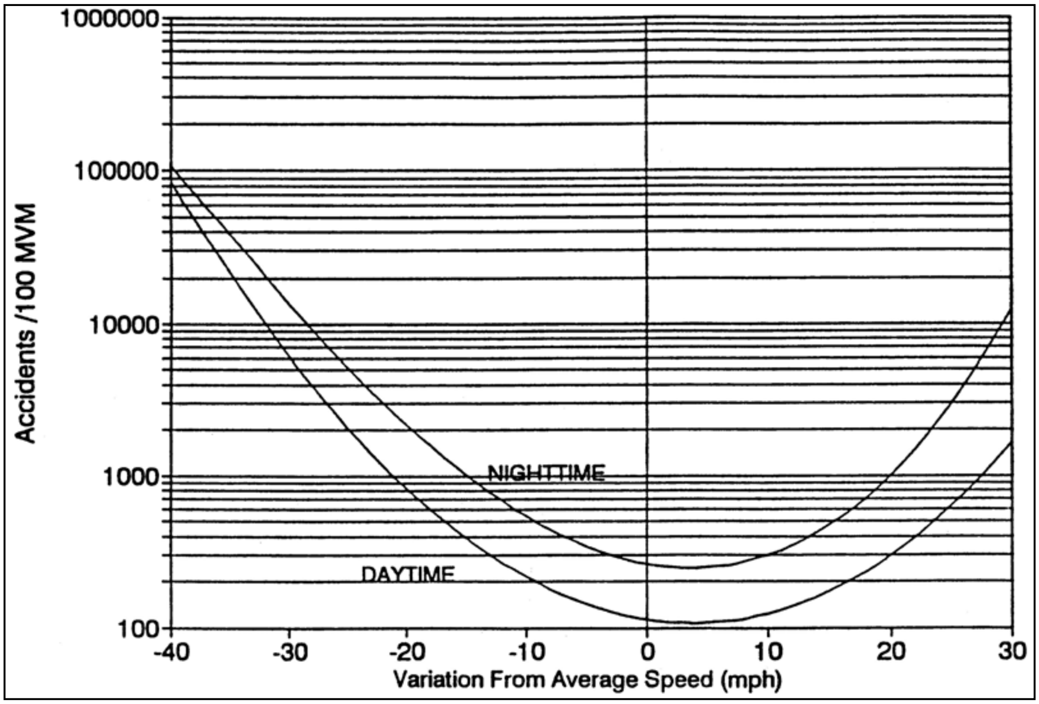


Figure 8.1 Accident Involvement Rate Due to Variation from Average Speed on Study Section, Daytime and Nighttime, Solomon (*14*)

The application of Solomon's accident involvement rate required the determination of the difference between the average speed of vehicles decelerating to enter the left-turn lane ($V_{LT, AVG}$) and the average speed of all traffic in the passing lane on a typical roadway segment (V_{RDWY}). $V_{LT, AVG}$ was calculated by averaging the upstream mean speed of left-turn vehicles ($V_{LT,TYP}$) and the mean speed of left-turn vehicles 50 ft in advance of the taper (V_{LT}).

Figure 8.2 shows the placement of detectors at each of the study sites and the detector identification numbers. The mean speed of left-turning vehicles (V_{LT}) for each of the study sites was determined during daytime and nighttime conditions from tracking the left-turning vehicles and calculating the mean speeds from the detector located 50 ft in advance of the beginning of the left-turn taper (Detector #9). The mean speed of left-turn vehicles on a typical roadway segment ($V_{LT,TYP}$) was assumed to be that of those relevant vehicles 1200 ft in advance of the beginning of the left-turn taper in the passing lane (Detector #14).

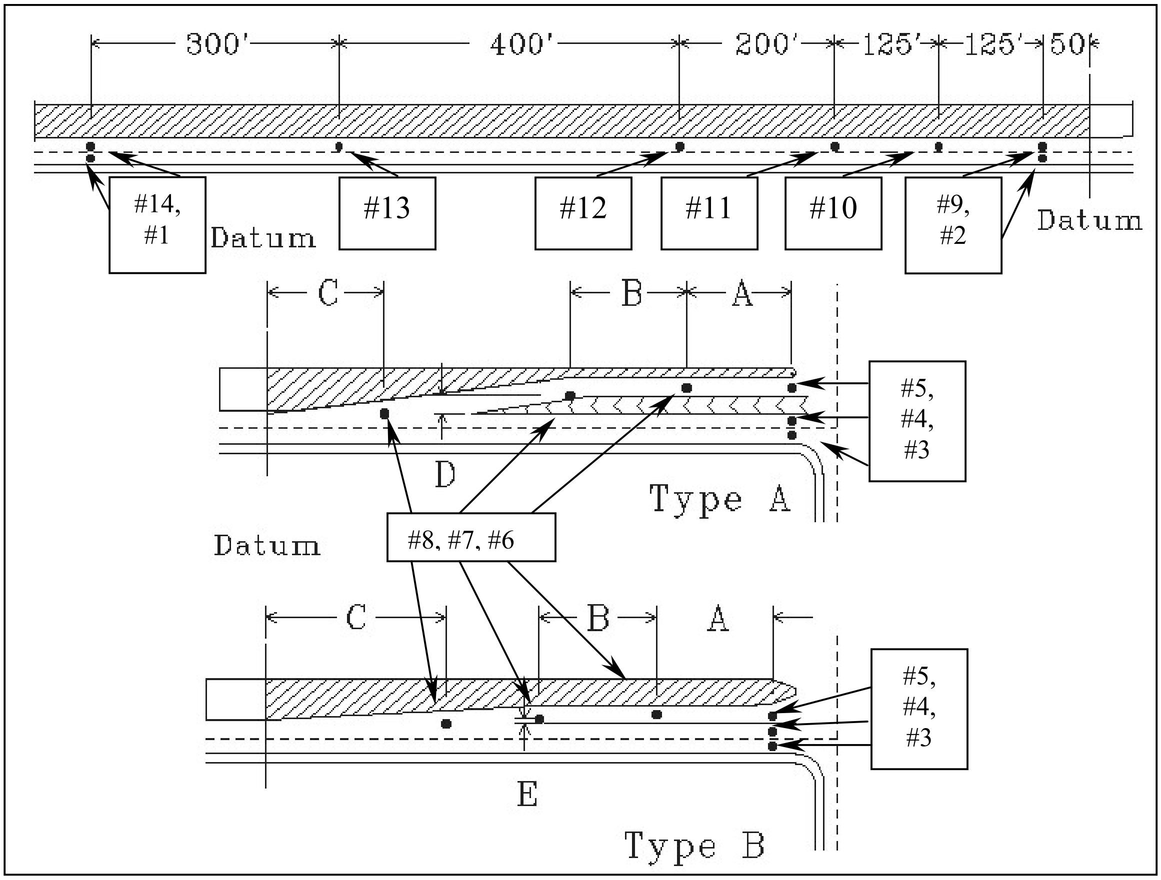


Figure 8.2 General Layout of NC-97 Detectors at Type A and B Median Breaks and Detector Identification Numbers.

At three of the study sites, data from Detector #13 (900 ft in advance of the taper) was used to determine the mean speed of $V_{LT,TYP}$ due to a malfunction in Detector #14. Due to mechanical difficulties with the speed detector synchronized timings, tracking of left-turn vehicles at Site 2 (55B) was not possible. The upstream mean speed of left-turn vehicles, $V_{LT,TYP}$ at the 55B site was estimated by the mean value of all vehicles at Detector #13. The mean speed of left-turn vehicles (V_{LT}) is approximated by using the mean speed of all vehicles from the location 175 ft in advance of the taper beginning (Detector #10) and the mean speed of only left-turn vehicles at the location 140 ft beyond the beginning of the taper (Detector #8) prorated for the distance that Detector #9 was between Detectors #8 and #10.

As mentioned in Chapter 3, a rigorous attempt was made to study intersections which were constructed exactly as the 1996 NDOR Roadway Design Manual depicted the Type A and B left-turn lanes (Figures 1.1, 1.2 and 1.3). No intersections were found that matched the 15 ft to 1 ft longitudinal-to-lateral taper along with the other required site criteria. Therefore, the data collected from the Type A and B left-turn lanes with tapers ranging from 5:1 to 20:1 was used to estimate $V_{LT,TYP}$, V_{LT}, and $V_{LT,AVG}$ at left-turn lanes with taper rates of 15:1.

Ratios of mean speed of left-turning vehicles at upstream locations to the longitudinal element of taper rate (the distance in feet longitudinally along the roadway to shift 1 foot laterally) for all study sites were plotted to determine a relationship between speed and taper rate for time of day. This relationship was also established for the mean speed of left-turning vehicles at the location 50 ft in advance of the taper (Detector #9). Figures 8.3 and 8.4 show this relationship for day/night conditions at the upstream (Detector #13 or #14) and Detector #9 locations, respectively. Distinguishing values between posted speed and left-turn type was not possible due to the limited number of study sites.

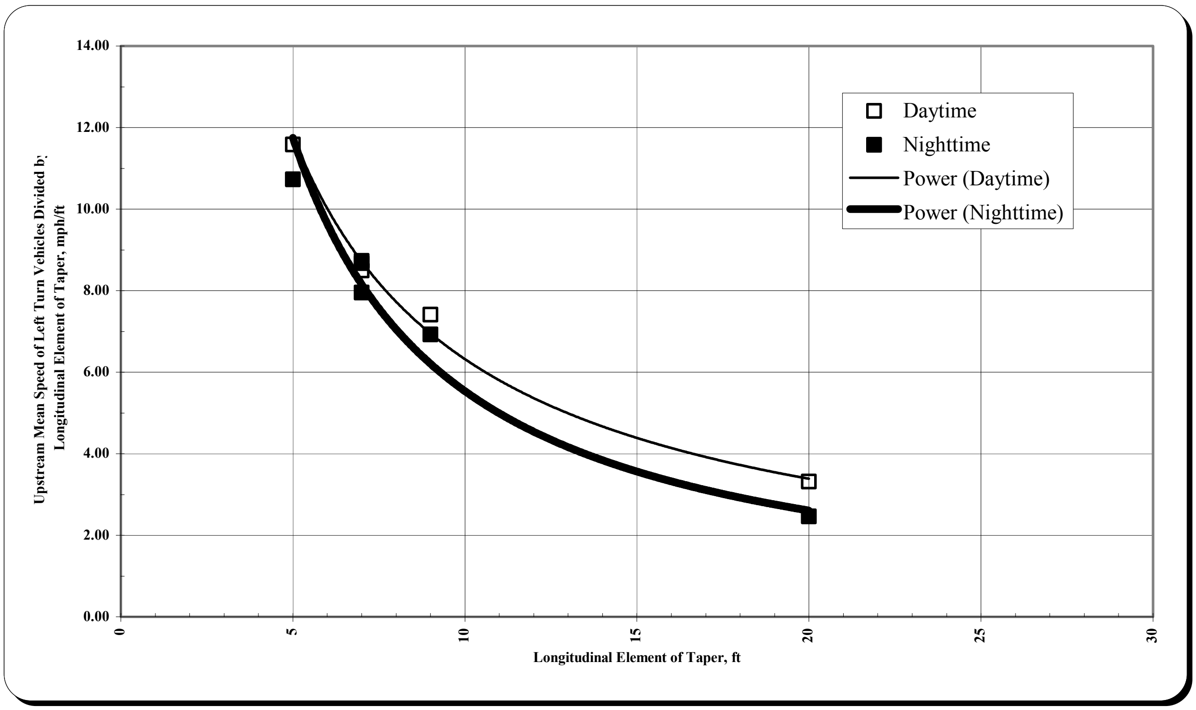


Figure 8.3 Upstream Mean Speed of Left-Turn Vehicles in the Passing Lane Divided by Longitudinal Taper versus Longitudinal Taper in the Daytime and Nighttime

Equations 8.1 and 8.2 were used to estimate the mean-speed-to-longitudinal-taper ratio for 15:1 left-turn lane tapers on rural expressways in the day and night respectively for left-turn vehicles in the passing lane. Estimated speeds for 15:1 tapers derived from the ratios are shown in Table 8.1 along with the actual mean speeds from study site detectors.

$$\text{RATIO}_{\text{UP,LT,DAY}} = 50.278\ T^{-0.9006} \qquad \textbf{(8.1)}$$

where,

$\text{RATIO}_{\text{UP, LT, DAY}}$ = upstream mean speed of left-turn vehicles divided by the longitudinal element of the left-turn lane taper rate in the daytime, mph/ft, and

T = longitudinal element of the left-turn lane taper rate, ft.

The adjusted R^2 value for Equation 8.1 is 0.99.

$$RATIO_{UP, LT, NT} = 67.518\ T^{-1.0863} \quad \textbf{(8.2)}$$

where,

$RATIO_{UP, LT, NT}$	= mean speed of left-turn vehicles at Detector #9 to the longitudinal element of the left-turn lane taper rate in the nighttime, ft, and
T	= longitudinal element of the left-turn lane taper rate, ft.

The adjusted R^2 value for Equation 8.2 is 0.98.

Equations 8.3 and 8.4 were used to estimate the mean-speed-to-longitudinal-taper ratio for 15:1 left-turn lane tapers on rural expressways in the day and night respectively for left-turn vehicles 50 ft in advance of the left-turn taper (Detector #9).

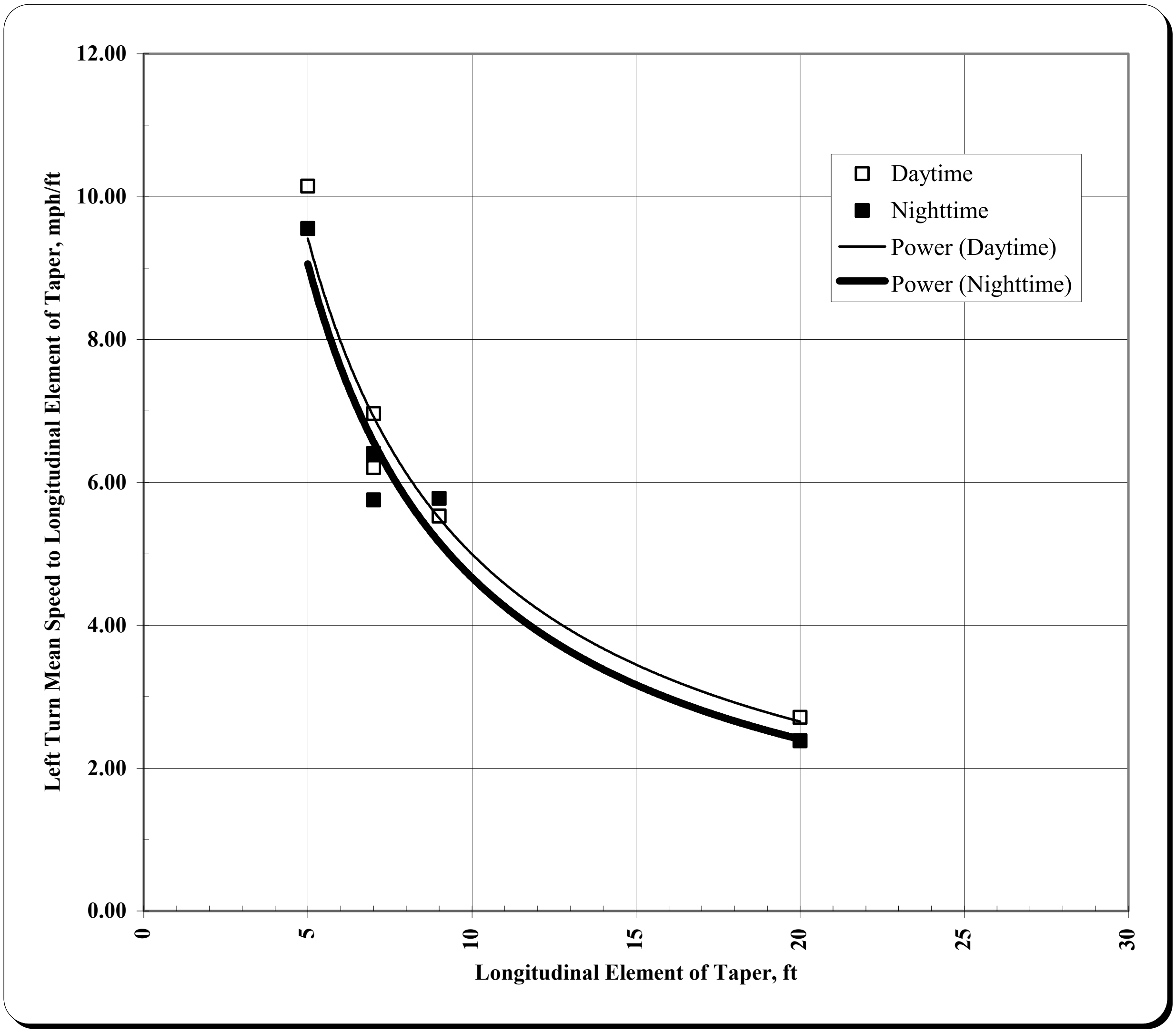


Figure 8.4 Mean Speed of Left-Turn Vehicles at Detector #9 (50 ft in Advance of the Taper) Divided by Longitudinal Element of Taper Rate versus Longitudinal Taper for Type A and B Left-Turn Types in the Daytime and Nighttime

Estimated speeds derived from the ratios are shown in Table 8.1 along with the actual mean speeds from study site detectors.

$$\text{RATIO}_{\text{DET \#9, LT, DAY}} = 40.986\ \text{T}^{-0.9140} \qquad \textbf{(8.3)}$$

where,

RATIO $_{\text{DET \#9, LT, DAY}}$ = mean speed of left-turn vehicles at Detector #9 (50 ft in advance of the taper) divided by the longitudinal element of the left-turn lane taper rate in the daytime, mph/ft and

T = longitudinal element of the left-turn lane taper rate.

The adjusted R^2 value for Equation 8.3 is 0.98.

$$\text{RATIO}_{\text{DET \#9, LT, NT}} = 42.247\ T^{-0.9568} \qquad \textbf{(8.4)}$$

where,

$\text{RATIO}_{\text{DET \#9, LT, NT}}$ = upstream mean speed of left-turn vehicles divided by the longitudinal element of the left-turn lane taper rate in the nighttime, mph/ft and

T = longitudinal element of the left-turn lane taper rate.

The adjusted R^2 value for Equation 8.4 is 0.97.

Table 8.1 Average Speed of Left-Turn Vehicles in the Passing Lane

Posted Speed/Left-Turn Type Category	Time of Day	Upstream Detector Number	Upstream Mean Speed of Left-Turn Vehicles, $V_{LT,TYP}$, mph	Mean Speed of Left-turn Vehicles, Detector #9, V_{LT}, mph	Average of Upstream and Left-Turn Speed, $V_{LT,AVG}$, mph
55A	Day	#14	59	49	54
55A	Night	#14	56	45	50
55B	Day	#13	58 [1]	51 [1]	54 [1]
55B	Night	#13	54 [1]	48 [1]	51 [1]
65A Davey	Day	#13	61	43	52
65A Davey	Night	#13	61	40	50
65A Firth	Day	#14	67	**50**	58
65A Firth	Night	#14	62	52	57
65B	Day	#13	66	54	60
65B	Night	#13	49	48	48
(15:1) [2,4]	Day	N.A.	66 [2]	52 [4]	59 [2,4]
(15:1) [3,5]	Night	N.A.	53 [3]	47 [5]	50 [3,5]

1 Values are estimated due to mechanical difficulties in the speed detectors during the 55B study preventing the tracking of left-turn vehicles.
2 Values are estimated from Equation 8.1
3 Values are estimated from Equation 8.2
4 Values are estimated from Equation 8.3
5 Values are estimated from Equation 8.4

The mean speed of all vehicles on a typical roadway segment (V_{RDWY}) was assumed to be that of those vehicles 1200 ft in advance of the beginning of the left-turn taper in the passing lane (Detector #14). As stated above, in three study site cases, data from Detector #13 was used (900 ft in advance of the taper) due to a malfunction in Detector #14 during the studies. Ratios of the mean speed of all vehicles at upstream locations to the longitudinal element of taper rate for all study sites were plotted to determine a relationship between speed and taper rate for left-turn type and time of day, similar to what was done previously for the left-turning vehicles.

The mean speeds that would be expected for 15:1 taper rates were estimated using the same method that was used to estimate left-turn mean speeds. Figure 8.5 shows the relationship between the upstream mean-speed-of-all-vehicles/longitudinal taper ratio and taper for day and night conditions.

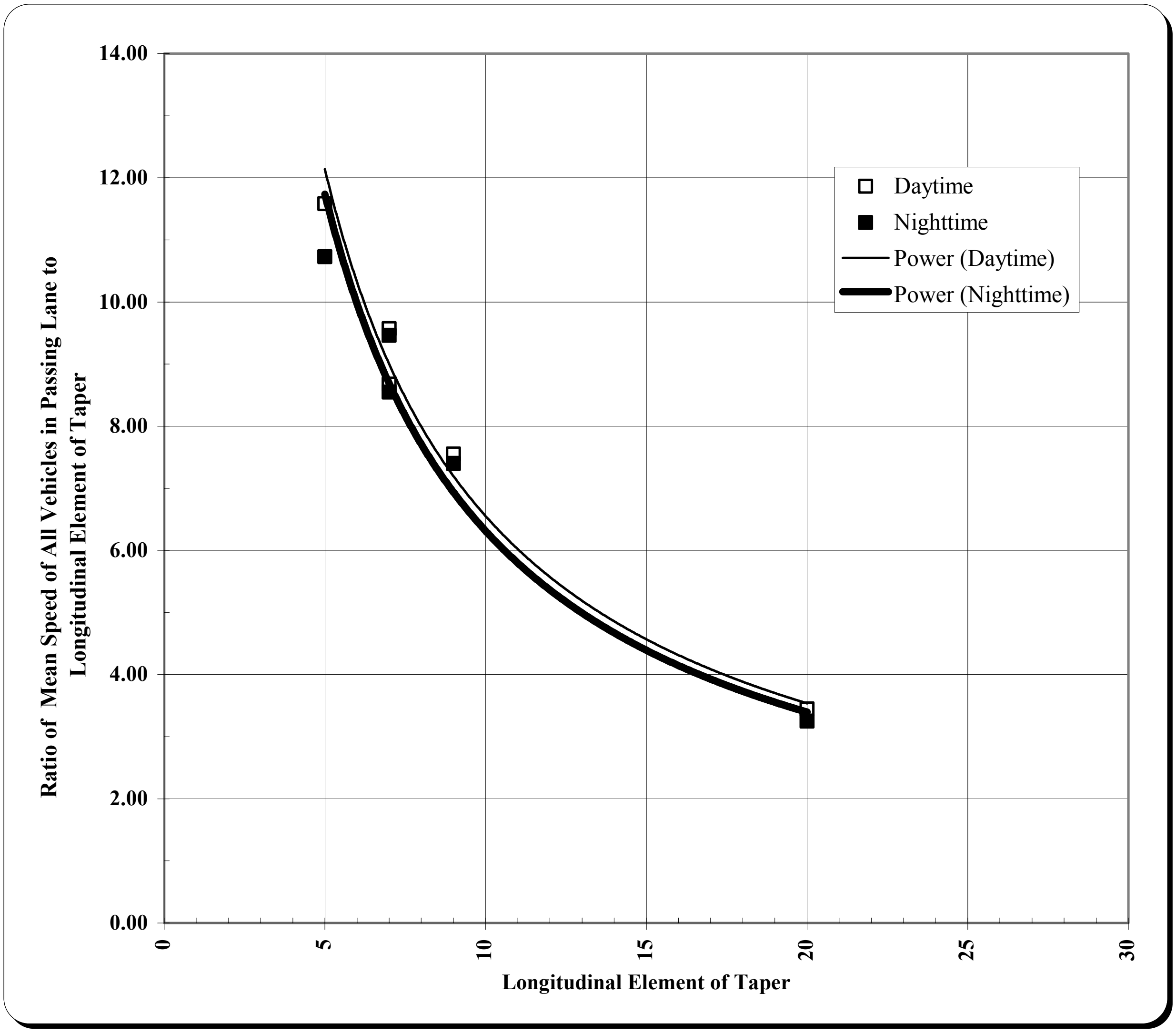


Figure 8.5 Upstream Mean Speed of All Vehicles Divided by Longitudinal Taper versus Longitudinal Taper for Type A and B Left-Turn Types in the Daytime and Nighttime

Equations 8.5 and 8.6 were used to estimate the mean-speed-to-longitudinal-taper ratio for 15:1 left-turn lane tapers on rural expressways for all upstream vehicles in the day and night respectively. Estimated speeds for left-turn lanes with 15:1 tapers derived from the ratios are shown in Table 8.2 along with the actual mean speeds from study site detectors.

$$\text{RATIO}_{\text{UP, ALL, DAY}} = 50.784\ T^{-0.8893} \quad \textbf{(8.5)}$$

where,

$\text{RATIO}_{\text{UP, ALL, DAY}}$ = upstream mean speed of all vehicles divided by the longitudinal element of the left-turn lane taper rate in the daytime, mph/ft, and

T = the longitudinal element of the left-turn lane taper rate, ft.

The adjusted R^2 value for Equation 8.5 was 0.98.

$$\text{RATIO}_{\text{UP, ALL, NT}} = 49.539\ T^{-0.8947} \qquad \textbf{(8.6)}$$

where,

$\text{RATIO}_{\text{UP, ALL, NT}}$ = upstream mean speed of all vehicles divided by the longitudinal element of the left-turn lane taper rate in the nighttime, mph/ft and

T = the longitudinal element of the left-turn lane taper rate.

The adjusted R^2 value for Equation 8.6 was 0.98.

Table 8.2 Mean Speed of All Vehicles in the Passing Lane on a Typical Roadway Segment

Posted Speed/Left-Turn Type Category	Time of Day	Upstream Detector Number	Upstream Mean Speed of All Vehicles, V_{RDWY}, mph
55A	Day	#14	61
55A	Night	#14	60
55B	Day	#13	58
55B	Night	#13	54
65A Davey	Day	#13	67
65A Davey	Night	#13	66
65A Firth	Day	#14	68
65A Firth	Night	#14	67
65B	Day	#13	69
65B	Night	#13	65
15:1	Day	N.A.	69[1]
15:1	Night	N.A.	66[2]

[1] Estimated from Equation 8.5
[2] Estimated from Equation 8.6

The average roadway speed (V_{AVG}) is the average speed of all vehicles in the passing lane on the roadway in a typical roadway segment, including both left-turning and non-left-turning vehicles. The average roadway speed was computed using the Equation 8.7.

$$V_{AVG} = P_{LT}\, V_{LT,AVG} + (1 - P_{LT})\, V_{RDWY} \qquad \textbf{(8.7)}$$

where,

V_{AVG} = average roadway speed in the passing lane, mph,
P_{LT} = portion of left-turn vehicles,
$V_{LT,AVG}$ = average speed of left-turning vehicles, mph, and
V_{RDWY} = roadway speed on a typical highway segment in the passing lane, mph.

The average roadway speeds computed for the left-turn percentages from 0 to 35 percent in 5 percent increments are shown in Table 8.3. These average roadway speeds provided the zero variation point used to estimate the accidents associated with left-turn deceleration in the passing lane using Solomon's findings. They reflected the differences between the average speed of left-turning vehicles ($V_{LT,AVG}$ from Table 8.1) and the mean speed of all vehicles for each category (V_{RDWY} from Table 8.2). For example, the 55A-Day average roadway speed for a mean roadway speed (V_{RDWY}) of 60.72, left-turn mean speed of 54.09 and 15 percent left turns would be (0.15)(54.09) + (1 – 0.15)(60.72) or 59.73 mph.

Table 8.3 Average Roadway Speed, V_{AVG}, mph used as the Zero Speed Variation Point to Determine Accident Involvement Rate from Solomon's Findings

Speed/Type Category	Time of Day	Percent Left Turns							
		0	**5**	**10**	**15**	**20**	**25**	**30**	**35**
55A	Day	61	60	60	60	59	59	59	58
55A	Night	60	59	59	58	58	57	57	56
55B*	Day	58	58	58	57	57	57	57	57
55B*	Night	54	54	53	53	53	53	53	53
65A Davey	Day	67	66	66	65	64	63	63	62
65A Davey	**Night**	66	65	65	64	63	62	62	61
65A Firth	Day	68	67	67	67	66	66	65	65
65A Firth	Night	67	66	66	65	65	64	64	63
65B	Day	69	68	68	68	67	67	66	66
65B	Night	65	64	63	63	62	61	60	59
(15:1)*	Day	69	68	68	67	67	66	66	65
(15:1)*	Night	66	65	64	64	63	62	61	60

* Values are estimated using means from Table 8.1 and 8.2.

To determine an accurate estimate for the number of accidents occurring annually due to left-turn decelerating vehicles in the passing lane, the distance over which the left-turn vehicles are assumed to decelerate must be established. The left-turn deceleration distance is the distance from the detector where an appreciable reduction in mean speed of the left-turn vehicles occurs to the location 50 ft in advance of the taper (Detector #9). Table 8.4 shows details of left-turn vehicle mean speeds between 1200 and 50 ft in advance of the taper and the resulting deceleration distance. The deceleration distance for 15:1 tapered left-turn lanes in day/night conditions was also estimated to be 450 ft.

Table 8.4 Determination of Deceleration Influence Length of Left-Turn Vehicles in the Passing Lane from Mean Speed of Left-Turn Vehicles

Posted Speed/ Type Category	Time of Day	Mean Speed Detector #14, 1200 ft from taper	Mean Speed Detector #13, 900 ft from taper	Mean Speed Detector #12, 500 ft from taper	Mean Speed Detector #11, 300 ft from taper	Mean Speed Detector #10, 175 ft from taper	Mean Speed Detector #9, 50 ft from taper	Decele-ration Influence Length, ft
55A	Day/ Night	59	59	60	57	52	49	**450**
55B	Day/ Night	N. A.	N. A.	N. A.	N. A.	N. A.	N. A.	**450***
65A Davey	Day/ Night	N. A.	61	60	56	N.A.	43	**450**
65A Firth	Day/ Night	67	64	65	57	53	50	**450**
65B	Day/ Night	N.A.	66	64	60	54	53	**450**
15:1	Day/ Night	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	**450***

* Estimated value

Although some accidents involving left-turn vehicles may occur in the driving (right approach through) lane of rural expressways, accidents were assumed to occur exclusively in the passing (left approach through) lane for the purpose of calculations in this study. Therefore, to determine the number of left-turn vehicles in the passing lane traffic stream that cause accidents due to excessive speed differentials, the expressway AADT values were:

1) divided by 2, assuming that the directional split on the expressway is equal,
2) multiplied by the portion of the total directional volume in the passing lane, and
3) multiplied by the portion of left-turning vehicles in the total directional passing stream.

Table 8.5 shows lane volume portions collected from detector vehicle counts used for the different posted-speed/type categories. The 55-mph posted-speed types had more volume distributed in the passing lane, which would be likely since lower posted speeds are used at unsignalized rural locations near the fringe of cities. The three 65-mph posted-speed sites had a passing lane volume of about 22 percent of the total approach volume. Since the posted-speed/type categories exhibited different portions of total traffic volumes in the passing lane of the expressway, these portions were also used to represent the portion of total traffic volume for estimating 15:1 left-turn lane conditions.

Table 8.5 Distribution of Traffic Volume on Rural Expressway Through Lanes at Study Sites

Speed/ Type Category	Driving Lane (Right Approach Through) Volume, Portion of Total Volume	Passing Lane (Left Approach Through) Volume, Portion of Total Volume	Portion of Total Volume Used for Accident Calculation, P_{PL}[a]
55A	0.663	0.337	0.38
55B	0.580	0.420	0.38
65A Davey	0.784	0.216	0.22
65A Firth	0.778	0.222	0.22
65B	0.790	0.210	0.22
55A(15:1)	N.A.	N.A.	0.38*
55B(15:1)	N.A.	N.A.	0.38*
65A(15:1)	N.A.	N.A.	0.22*
65B(15:1)	N.A.	N.A.	0.22*

[a] Values determined by averaging the portion of passing lane volumes for each posted speed category.
* Assumed values

Number of Accidents

The estimated number of accidents per year caused by vehicles decelerating in the through traffic passing lane for each posted-speed/left-turn type was calculated using the Equation 8.8.

$$ACC_{55/65,A/B} = [(P_{Day})(I_{Day,\,55/65,A/B}) + (P_{Night})(I_{Night,55/65,A/B})][P_{LT}][P_{PL}][AADT_{EXPWY} \div 2][L_{DEC}][365] \tag{8.8}$$

$ACC_{55/65,A/B}$ = annual number of accidents caused by left-turn decelerating vehicles in the passing lane of the rural expressway for posted speed/left-turn type,

P_{Day} = portion of daytime traffic,

$I_{Day,\,55/65,A/B}$ = daytime accident involvement rate for posted speed/left-turn type, accidents per 100 million vehicle miles,

P_{Night} = portion of nighttime traffic,

$I_{Night,\,55/65.A/B}$ = nighttime accident involvement rate for posted speed/left-turn type, accidents per 100 million vehicle miles,

P_{LT} = portion of left-turning vehicles,

P_{PL} = portion of total traffic in passing lane (55 mph posted speed = 0.38, 65 mph posted speed = 0.22),

$AADT_{EXPWY}$ = annual average daily traffic of the rural expressway, vehicles per day, and

L_{DEC} = left-turn deceleration distance, miles (450 ft (0.0852 miles) for all sites).

A dimensional analysis is shown below to further detail how Equation 8.9 (simplified version of Equation 8.8) was determined.

Dimensional analysis:

$$\frac{\text{accidents}}{\text{year}} = \left[\frac{(\text{none})(\text{accidents})}{1 \text{ x } 10^8 \cancel{\text{veh-miles}}} + \frac{(\text{none})(\text{accidents})}{1 \text{ x } 10^8 \cancel{\text{veh-miles}}}\right] [\text{none}]\ [\text{none}]\ \left[\frac{\cancel{\text{veh}}}{2\ \cancel{\text{day}}}\right] [\cancel{\text{mile}}]\ \left[\frac{\cancel{\text{days}}}{\text{year}}\right]$$

The portions of daytime and nighttime traffic used in Equations 8.8 and 8.9 are the averages of those found for continuous traffic counting stations on rural expressway sections of the expressway system in the state of Nebraska (23). On average, 73 percent of the daily traffic occurs during the daytime (considered as 6:01 to 18:00) and 27 percent occurs at night (considered 18:01 to 6:00 hours).

The daytime and nighttime accident involvement rates were determined from the relationship between speed differential and accidents established by Solomon shown in Figure 8.1. The average roadway speeds (V_{AVG}) from Table 8.3 were considered as the zero "variation from average speed" location in Figure 8.1 and the speed distributions from the detector located 50 ft in advance of the left-turn lane taper (Detector #9) from each of the posted-speed/type/time-of-day categories were used to establish weighted involvement rates that would most closely estimate left-turn decelerations at expressway intersections. A normal speed distribution was assumed for the 55B option since Type 55A and 65B sites resulted in normal distributions. The speed differential frequencies for the 55B option were assumed to be similar to those exhibited at the 55A site. Speed distributions for the 15:1 left-turn tapers were assumed to be of the same type as the corresponding posted-speed/type at the study sites. There was a significant difference between the speed distributions of the left-turning vehicles at the 65A-Davey and 65A-Firth sites, so a combined speed distribution was not used to represent the posted-speed/type category. The 65A speed distribution from the Firth study site was used for the 65A(15:1) left-turn type since it would result in more conservative accident reductions. The summation of the normalized accident involvement rate per speed differential was taken to find the total rate for each left-turn percentage from 0 to 35 for both day and night. These values are shown in Tables 8.6 and 8.7.

Table 8.6 Daytime Accident Involvement Rates Resulting from Field Data

Study Site	**Speed/Type Category**	**Percent Left Turns**							
		0	**5**	**10**	**15**	**20**	**25**	**30**	**35**
1	55A	681	723	769	817	871	928	989	1054
2	55B	503	519	535	551	569	587	605	625
3	65A-Davey	3790	4725	5805	7008	8334	8748	11200	12677
4	65A-Firth	5368	5750	6145	6543	6960	7387	7813	8255
5	65B	837	1018	1106	1203	1311	1427	1554	1682

Table 8.7 Nighttime Accident Involvement Rates Resulting from Field Data

Study Site	Speed/Type Category	Percent Left Turns							
		0	5	10	15	20	25	30	35
1	55A	864	1040	1115	1202	1297	1402	1517	1644
2	55B	575	585	596	606	618	629	641	653
3	65A-Davey	4233	4937	5765	6723	7853	9164	10661	12412
4	65A-Firth	6040	6415	6812	7214	7627	8051	8485	8929
5	65B	1188	1356	1551	1785	2057	2384	2765	3220

Equation 8.8 with known variables for Nebraska included is shown below:

$$ACC_{55/65,A/B} = \frac{[(0.73)(I_{Day,\ 55/65,A/B})+(0.27)(I_{Night,55/65,A/B})]\ [P_{LT}][P_{PL}][AADT_{EXPWY}][0.0852][365]}{2(100{,}000{,}000)}$$

Equation 8.9 is a simplified version of Equation 8.8 **(8.9)**

$$ACC_{55/65,A/B} = 1.555 \times 10^{-7}[(0.73)(I_{Day,\ 55/65,A/B}) + (0.27)(I_{Night,55/65,A/B})]\ [P_{LT}]\ [P_{PL}][AADT_{EXPWY}]$$

Table 8.8 shows characteristics of speed differentials for all study site categories at the location 50 ft in advance of the taper (Detector #9) and Figure 8.6 shows the frequency percentages for speed differential at the 55A, 65A Davey, 65A Firth, and 65B sites during the daytime.

Table 8.8 Speed Differential Characteristics 50 Ft in Advance of the Left-Turn Taper

Speed/Type Category	Time of Day	Speed Distribution Type	Number of Left-Turn Vehicles	Difference Between Mean Speeds of V_{AVG} and $V_{LT,AVG}$, mph
55A	Day	Normal	142	5
55A	Night	Normal	4	8
55B	Day	Normal [1]	127 [2]	-- [3]
55B	Night	Normal [1]	17 [2]	-- [3]
65A Davey	Day	Gamma	45	11
65A Davey	Night	Gamma	7	13
65A Firth	Day	Gamma	44	8
65A Firth	Night	Gamma	9	8
65B	Day	Normal	82	7
65B	Night	Normal	18	11

1 Assumed speed distribution
2 Values taken from Detector #5.
3 Not available due to detector malfunction.

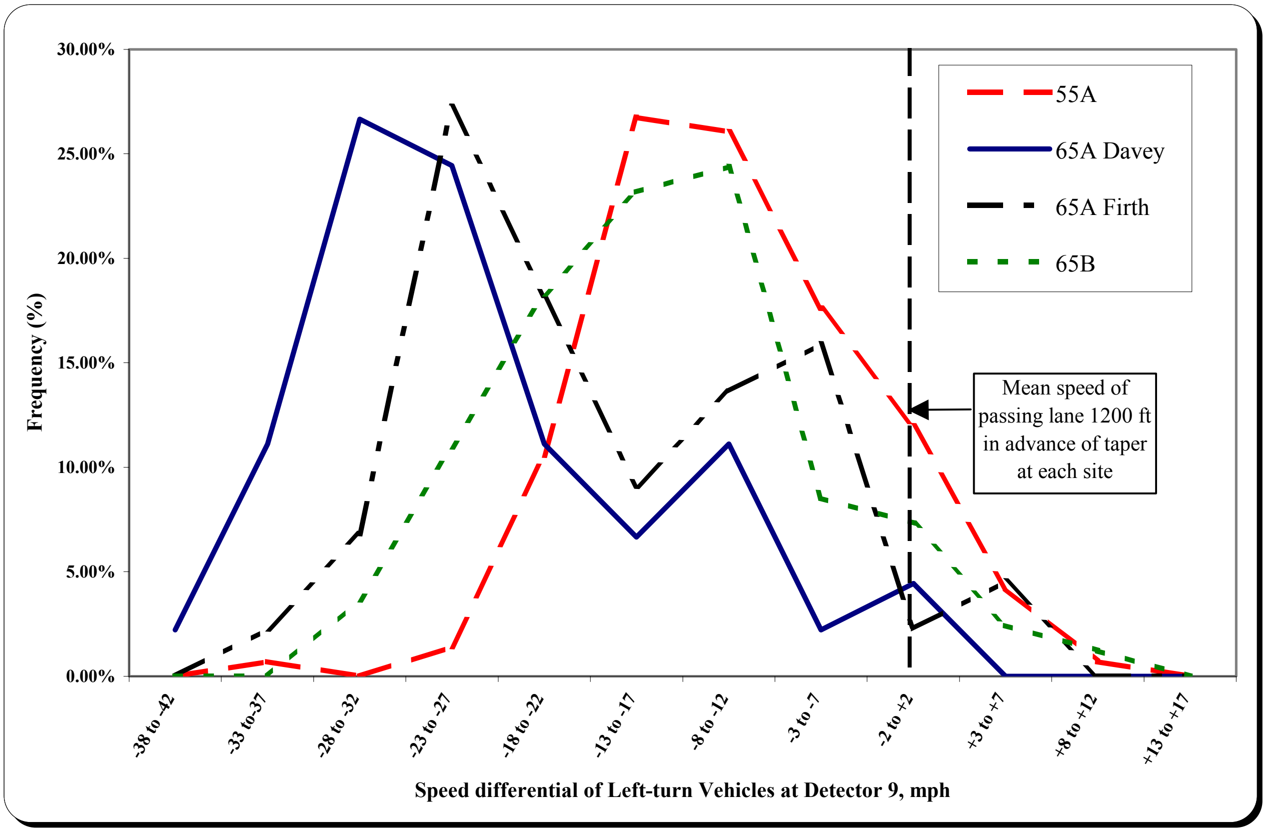


Figure 8.6 Speed Differential of Left-Turn Vehicles 50 ft in Advance of the Taper

The 55A site has the "safest" distribution of speeds around its mean, according to Solomon (*14*). The distribution shape is normal in nature and the difference between the mean of the left-turn vehicles and the through vehicles is 5 mph in the daytime and 8 mph at night with a modal speed of about 15 mph below the mean of the through traffic. According to the 2001 Green Book, a 10-mph speed reduction of left-turn vehicles in the through lane is acceptable (*4*). The 65B site also has a normal distribution shape but exhibits higher differences in mean speed (7 mph in the day and 11 mph at night) and similar modal values of speed reduction (14 mph less than the daytime mean and 18 mph less than the nighttime mean). Both the 65A Davey and 65A Firth sites exhibit a gamma distribution shape with the peak of the distribution about 30 mph and 25 mph below the mean, respectively. This indicates that left-turning drivers are decelerating more in the passing lane before entering the left-turn lane.

Both 65A sites exhibit an increased speed differential that is lower than the mean speed of the average roadway segment. The shift is more pronounced for the Davey site than the Firth site. Review of the available data at these two sites indicated the obvious difference between them is that the narrow 8-ft median between the offset left-turn lane and the opposing driving lane shoulder is surfaced at the Davey site and unsurfaced at the Firth site. Photographs taken at the location 50 ft in advance of the taper from the Davey and Firth intersections are shown in Figure 8.6. The taper rate at the Davey site is 7:1 (longitudinal to lateral) and the taper at the Firth site is 9:1.

65A – Highway 77 and Davey Road Detector #9

65A – Highway 77 and Firth Road Detector #9

Figure 8.7 Photographs of Type A-65 Intersections from 50 ft in Advance of the Taper

Drivers may be assuming that the intersection is much closer to the beginning of the taper than it actually is due to the placement of the left-turn lane sign and the paved median, causing drivers to decelerate further in advance of the taper location and increasing their speed differential with through vehicles. This behavior is undesirable from a safety standpoint. The taper rates for the two sites were slightly different (7:1 for the Davey site and 9:1 for the Firth site). It was assumed that the additional 58 ft length of the Firth site would not be perceivable by left-turning drivers approaching the intersection at high speeds.

Sight Obstructions

The second of the four predominant safety issues addressed in this research study when comparing the Type A offset left-turn lane to the Type B traditional left-turn lane concerns sight obstructions caused by opposing left-turn vehicles. The geometric configuration of the Type A left-turn lane shown in the 1996 NDOR Roadway Design Manual has a positive offset on a typical intersection with a zero degree skew (perpendicular crossroad). A positive offset is defined as the situation where the right edge of the opposing left-turn lane is to the right of the left edge of the left-turn lane as shown in Figure 1.1. The positive offset effectively allows a left-turn vehicle to view the approaching traffic without obstruction from opposing left-turn vehicles. The geometric configuration of the Type B left-turn lane design has a negative offset with the opposing left-turn lane. A negative offset is defined as the situation where the right edge of the opposing left-turn lane is to the left of the left edge of the left-turn lane as shown in Figure 1.2 and 1.3. Therefore, if opposed, a driver wishing to turn left has a sight distance less than required for accomplishing a left-turn movement. An opposing vehicle at a Type B median break allows a longitudinal sight distance of 107 ft to the near opposing lane and 141 ft to the far opposing lane, assuming that the driver's eye is positioned 6 ft from the left edge of the left-turn lane and 8 ft back from the concrete island nose, the opposing vehicle is a design combination truck in the same position on the opposing left-turn lane, and the median opening length is 72 ft (typical condition shown in Figure 1.2 (*1*)). Figure 8.8 shows this limited sight distance situation. According to the 2001 Green Book minimum intersection sight distance model based on gap acceptance (*4, p. 678*), 725 ft of longitudinal sight distance is required for a passenger car making a left-turn movement across 25 ft of median plus an additional 12 ft through lane at an expressway design speed of 70 mph (equivalent of 7.05 seconds of critical gap). A distance of 992 ft is required for a worst-case scenario for a WB-62 combination truck (9.0 seconds of critical gap).

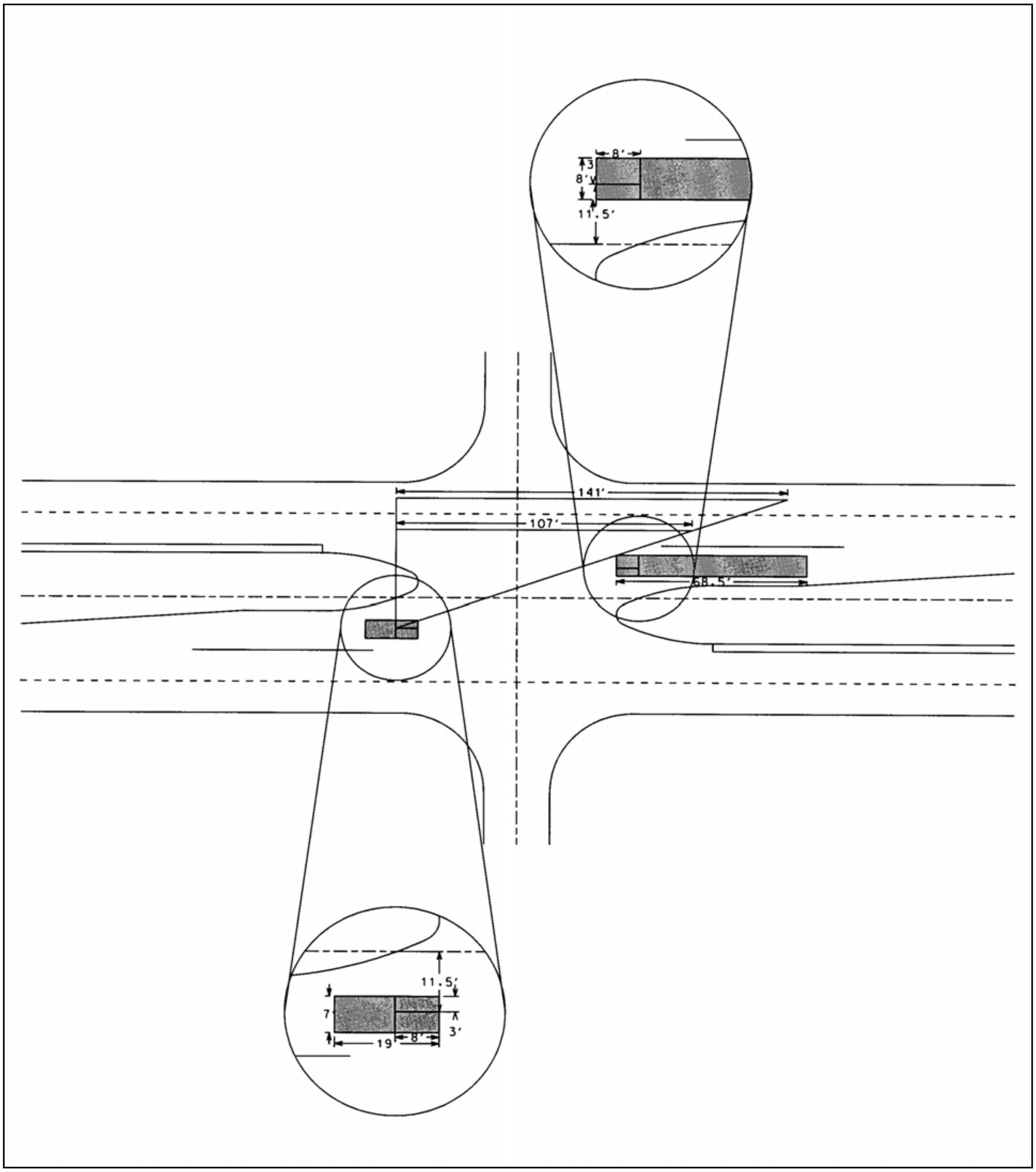


Figure 8.8 Sight Obstruction of Opposing Vehicle at Type B Median Break

An estimate of the annual accidents at two-way stop-controlled intersections was necessary to determine the percentage of accident reduction that could be expected from removing the opposing left-turn sight obstruction by using a Type A offset left-turn lane. An accident prediction model developed by Bonneson and McCoy (*25*) using a subset from the FHWA's Highway Safety Information System (HSIS) (*26*) was used to predict the number of accidents that would occur at rural expressway intersections with two-way stop-control on the minor roadway and is shown as Equation 8.10.

$$ACC_{EXP,TOTAL} = 0.6503(AADT_{EXPWY}/1000)^{0.2925}(AADT_{MINOR}/1000)^{0.7911} \qquad \textbf{(8.10)}$$

where,

$ACC_{EXP,TOTAL}$ = expected number of accidents at rural expressway intersections with two-way stop-control on the minor roadway due to left-turn-leaving accidents, per year,
$AADT_{EXPWY}$ = average annual daily expressway traffic volume, vehicles per day, and
$AADT_{MINOR}$ = average annual daily minor (cross) road traffic volume, vehicles per day.

A NDOR database including all unsignalized intersection accidents on rural expressways from the years 1988 to 2000 (*27*) was searched to determine the average volume of crossroad traffic at unsignalized intersections. Minor road traffic volume frequencies for 104 minor roads intersecting with expressways at unsignalized intersections having left-turn-leaving multi-vehicle accidents are shown in Figure 8.9.

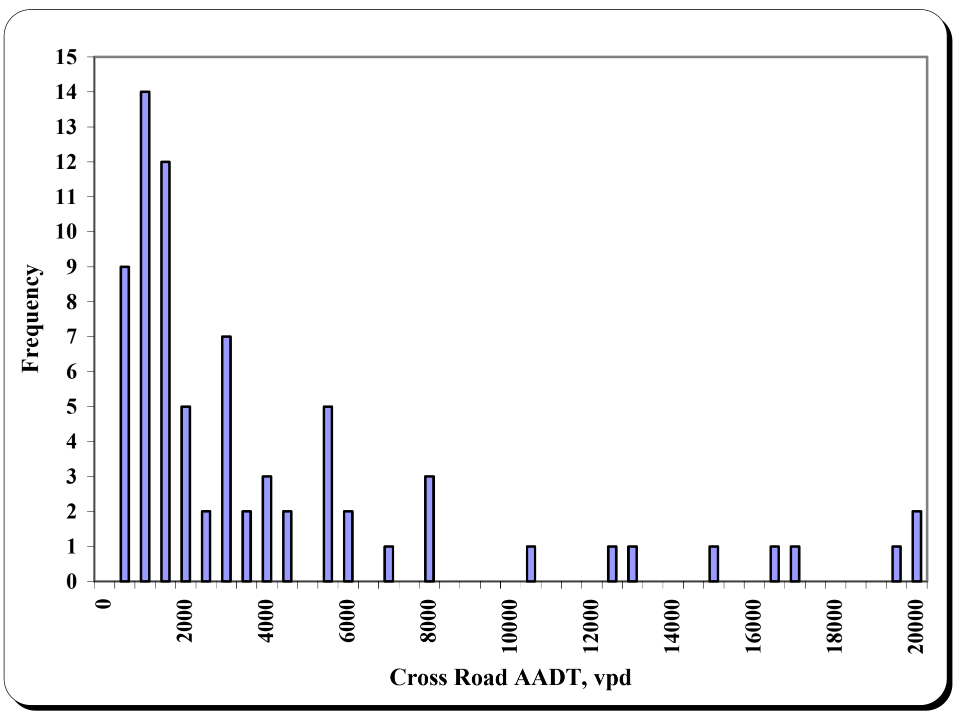


Figure 8.9 Minor Road Traffic Volume Frequencies for Unsignalized Expressway Intersections Experiencing Accidents from 1988-2000

The portion of left-turn-leaving accidents with respect to the total number of accidents at 208 unsignalized rural expressway intersections was also determined from the 13-year database. Major contributing human factors for this type of multi-vehicle collision were also tabulated. The accident severity results of the database search are shown in Table 8.9 and Table 8.10.

Table 8.9 Summary of Left-Turn Leaving Accident Severity on Unsignalized Expressway Intersections in Nebraska, 1988-2000

Accident Type	Total Accidents	Fatal	Injury			Property Damage Only
			Disabling	Visible	Possible	
Left-Turn-Leaving	207	3	14	51	40	99
All Accidents	3375	34	179	471	823	1868
Portion of Left-Turn-Leaving to Total Accidents	0.061	0.088	0.078	0.108	0.049	0.053

These values show that the left-turn-leaving type of accident results in more than its share of severe outcomes since the portions of fatal and disabling/visible injuries are greater than the portion of total accidents that the left-turn-leaving type represents.

Table 8.10 shows that the attributing cause of left-turn leaving accidents (as reported by the attending accident report authority) is failure to yield the right-of-way to the approaching vehicle.

Table 8.10 Summary of Potential Contributing Human Factors to Left-Turn-Leaving Accidents on Expressway Intersections in Nebraska, 1988-2000

Human Factor	Total	Fatal	Injury			Property Damage Only
			Disabling	Visible	Possible	
Failure to Yield	151	2	11	41	25	72
Other Improper Turn	6	0	0	0	2	4
Improper/ No Turn Signal	2	0	0	0	0	2

To quantify safety effects of Type A offset intersections, it was necessary to establish a percent reduction in accidents that could be expected by removing the sight obstruction of an opposing left-turn vehicle. No accident reduction factor is currently known to exist for this specific countermeasure on rural unsignalized expressway intersections. Jorgensen et al (*28*) documented the realization of a 50 percent reduction in left-turn crashes by providing a protected left-turn phase or removing a permissive left-turn movement at a signalized intersection which would have a similar effect of reducing the opportunity of a left-turn vehicle colliding with an opposing through vehicle. Mitchell (*29*) recognized a 67 percent accident reduction on before-and-after studies of 5 intersections in Concord, CA when problem sight obstructions were removed. After consideration of these three factors, a 50-percent reduction factor was used as a conservative value to quantify the decrease in societal accident costs realized by the use of Type A offset left-turn lanes.

The expected left-turn-leaving accidents associated with Type A and Type B left-turn lanes can be calculated using Equations 8.11 and 8.12 respectively.

$$ACC_{LT\text{-}T\text{-}LV,\ Type\ A} = A_{R,\ Type\ A}\,(P_{LT\text{-}T\text{-}LV})(ACC_{EXP,TOTAL}) \qquad \textbf{(8.11)}$$

$$ACC_{LT\text{-}T\text{-}LV,\ Type\ B} = A_{R,\ Type\ B}\,(P_{LT\text{-}T\text{-}LV})(ACC_{EXP,TOTAL}) \qquad \textbf{(8.12)}$$

where,

$ACC_{LT\text{-}T\text{-}LV,\ Type\ A}$ = expected number of left-turn-leaving accidents per year at intersections with Type A left-turn lanes,
A_R = accident reduction factor (0.50 for Type A, 1.00 for Type B),
$P_{LT\text{-}T\text{-}LV}$ = average portion of left-turn-leaving accidents as a part of the total number of accidents at rural expressway intersections (0.061),
$ACC_{EXP,TOTAL}$ = annual total expected accidents at rural expressway intersections, and
$ACC_{LT\text{-}T\text{-}LV,\ Type\ B}$ = expected number of left-turn-leaving accidents per year at intersections with Type B left-turn lanes,

Accident prediction values were used in Chapter 9 to compare accident costs associated with the use of a Type A or Type B left-turn lane.

Deceleration Rates and Behavior Patterns

The third predominant safety issue investigated by this research involved average deceleration rates within the left-turn taper and storage lane. Average deceleration rates of left-turners were calculated for each of the study sites to determine if there was any difference between Type A and Type B locations. Speed differences from the location 50 ft in advance of the taper (Detector #9) and the island nose (Detector #5) were used to calculate the deceleration rate. Passenger cars at initial speeds of 30-70 mph decelerate at a rate of 4.8 fps^2 with normal, nonskid braking and at a rate of 6.7 fps^2 from initial speed of 0-30 mph. Table 8-11 shows a comparison of average deceleration rates at the Type A and B study sites. The average daytime rate of the Type A left-turn sites was 4.1 fps^2 and the average Type B daytime rate was 5.1 fps^2. All rates were within the given reasonable tolerances above except for the daytime 65B rate. The average nighttime rate for Type A left-turners was 3.7 fps^2 and for Type B the average rate was 3.8 fps^2. All rates are within reasonable tolerances. The Type A left-turn lane generally appears to result in slightly lower deceleration rates within the left-turn lane.

Table 8.11 Comparison of Average Deceleration Rates at Type A and B Study Sites

Posted Speed/Type	Average Daytime Deceleration Rate of Left-Turners from 50 ft in Advance of Taper to Island Nose, fps^2	Average Nighttime Deceleration Rate of Left-Turners from 50 ft in Advance of Taper to Island Nose, fps^2
55A	4.1	3.3
65A-Davey	3.9	3.4
65A-Firth	4.2	4.5
55B	4.4*	3.6*
65B	5.7	4.0

* Speed values 50 ft in advance of the taper are estimated for left-turners.

The impact of a following vehicle upon the decelerating behavior of left-turners was also studied. Figure 8.10 and 8.11 show the speed profiles of left-turn vehicles at Type A and Type B left-turn lanes with following vehicles within a 15 second headway and without followers, respectively. Both figures show that left-turn vehicles are traveling at higher speeds at Type B locations than Type A sites. Type B left-turn vehicles also travel about 5 mph faster when followed by another vehicle.

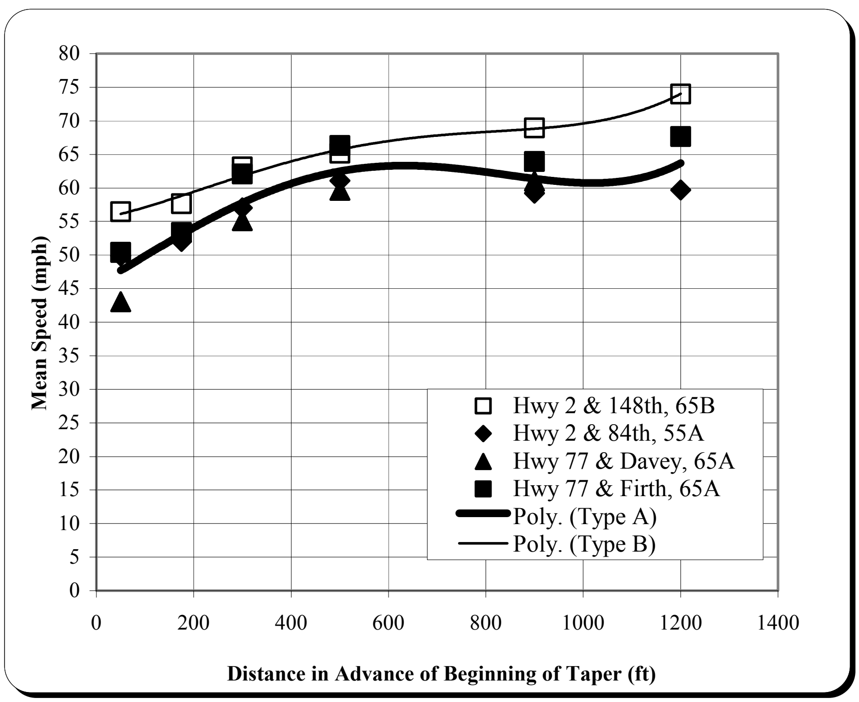


Figure 8.10 Speed Profile of Left-Turn Vehicles <u>with</u> a Following Vehicle within 15-Second Headways

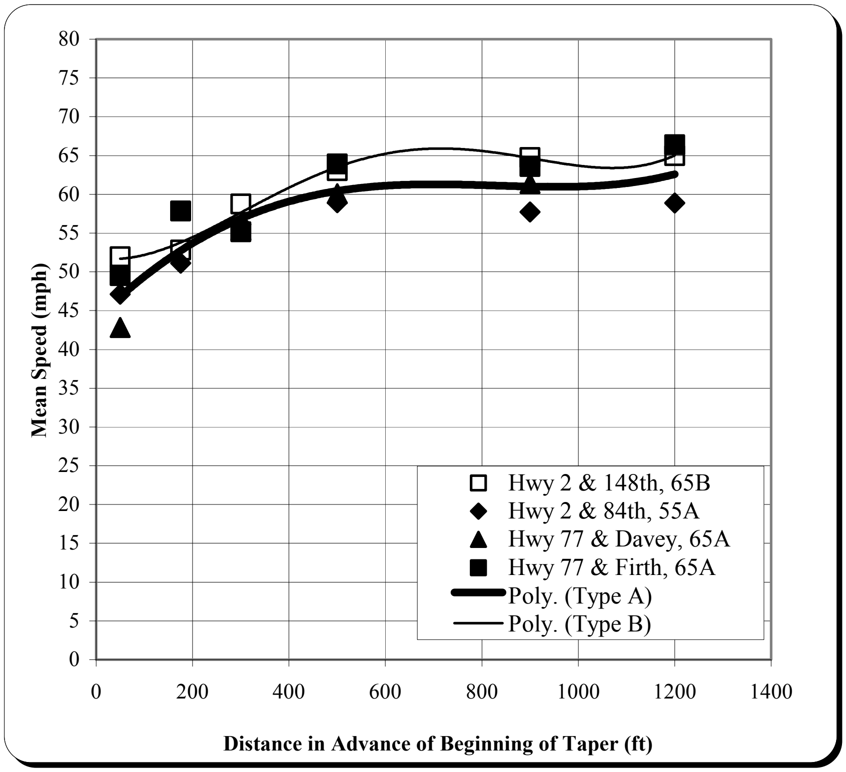


Figure 8.11 Speed Profile of Left-Turn Vehicles without a Following Vehicle within 15-Second Headways

Polynomial speed prediction equations were developed for left-turners with and without followers within 15-second headways for Type A and B left-turn types. Equations 8.13, 8.14, 8.15, and 8.16 along with their adjusted R^2 values are shown below.

$$V_{A,WF} = (6.0 \times 10^{-11})\, x^4 - (1.0 \times 10^{-7})\, x^3 + (3.0 \times 10^{-5})\, x^2 + 0.0406x + 45.615 \qquad \textbf{(8.13)}$$

where,

$V_{A,WF}$ = left-turn mean speed with followers within 15-second headways at Type A left-turn lanes, mph, and

x = distance from beginning of taper, ft.

Adjusted R^2 value for Equation 8.13 = 0.80

$$V_{B,WF} = (4.0 \times 10^{-11})\, x^4 - (9.0 \times 10^{-8})\, x^3 + (5.0 \times 10^{-5})\, x^2 + 0.0143x + 55.288 \qquad \textbf{(8.14)}$$

where,

$V_{B,WF}$ = left-turn mean speed with followers within 15-second headways at Type B left-turn lanes, mph, and

x = distance from beginning of taper, ft.

Adjusted R^2 value for Equation 8.14 = 0.98

$$V_{A,WOF} = (9.0 \times 10^{-12})\, x^4 + (1.0 \times 10^{-8})\, x^3 - (6.0 \times 10^{-5})\, x^2 + 0.0612x + 43.916 \quad \textbf{(8.15)}$$

where,

$V_{A,WOF}$ = left-turn mean speed without followers within 15-second headways at Type A left-turn lanes, mph, and

x = distance from beginning of taper, ft.

Adjusted R^2 value for Equation 8.15 = 0.81

$$V_{B,WOF} = (1.0 \times 10^{-10})\, x^4 - (2.0 \times 10^{-7})\, x^3 + 0.0002x^2 - 0.0105x + 51.840 \quad \textbf{(8.16)}$$

where,

$V_{B,WOF}$ = left-turn mean speed without followers within 15-second headways at Type B left-turn lanes, mph, and

x = distance from beginning of taper, ft.

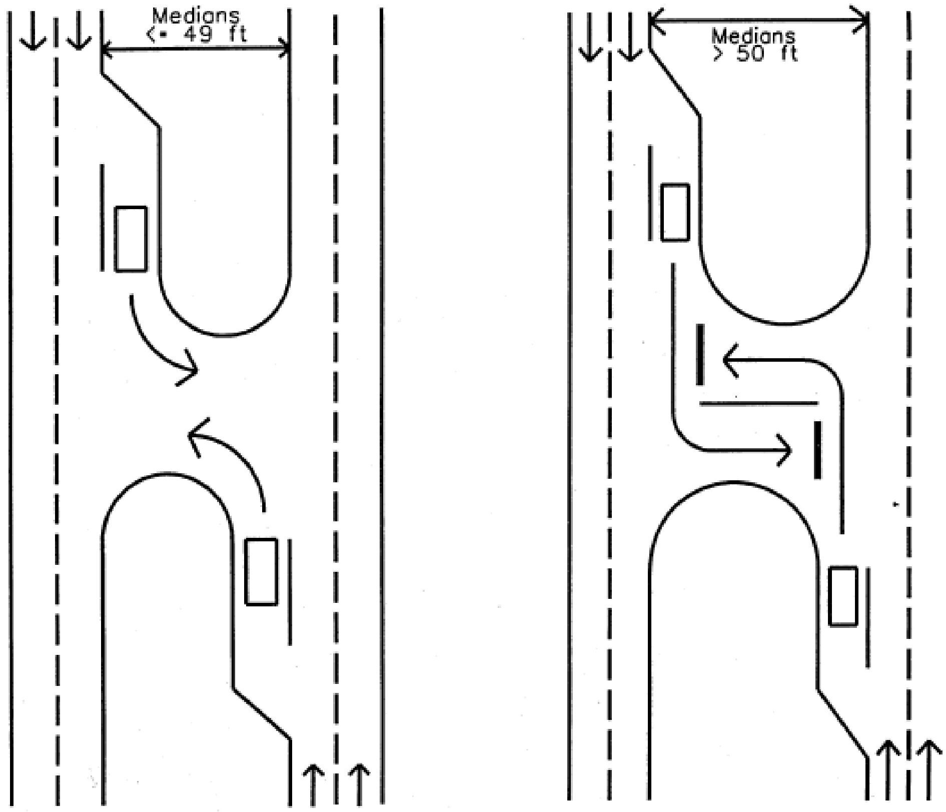
Adjusted R^2 value for Equation 8.16 = 0.99

It appears that the Type A left-turn design results in a lower overall mean vehicle speed in the passing lane in advance of the left-turn lane taper, whether or not the left-turner is followed by another vehicle. This fact would indicate that the Type A left-turn design may have more rear-end accidents due to large speed differentials with the through vehicles in the passing lane.

Driver Expectation Concerning Turning Paths

The last predominant safety issues addressed in this research when comparing the Type A offset left-turn lane to the Type B traditional left-turn lane concerns driver expectation related to the turning path chosen within the paved median area as drivers complete their left-turn movement.

In narrow medians with left-turn lanes or narrow median openings, left-turning vehicles turn in front of each other. On the other hand, at wide medians where the paved median area normally functions like a short street between opposing through-vehicle directions, left-turn vehicles usually turn behind each other. These left-turning behaviors are illustrated in Figure 8.12.



Turn-In-Front Behavior
Medians < 50 ft

Turn-Behind Behavior
Medians ≥ 50 ft

Figure 8.12 Left-turn Behaviors at Median Openings (9)

Field observations have found the turn-in-front behavior is predominant for intersection median widths less than 50 ft wide, and the turn-behind behavior is predominant for intersections with medians greater than or equal to 50 ft wide. Since the 40-ft median used by Nebraska is near the behavior-change value of 50 ft, drivers may be confused occasionally by the width of the paved intersection area. Although turn-behind behavior was not witnessed at any of the field study sites of Type A or Type B intersections, the Type A offset design would shift the driver laterally toward the opposing driving lanes and effectively reduce the paved area available to complete the left-turning maneuver. This reduction in pavement area may reduce the potential for a rare accident resulting from driver confusion, since turn-in-front behavior should be reinforced by effectively narrowing the median. No data was collected during this research project to support or discredit this assumption and no attempt was made to quantify an accident reduction value since frequency with which such a misperception occurs is unknown.

Chapter 9
COSTS OF LEFT-TURN LANE TYPES ON NEBRASKA EXPRESSWAYS

The development of a procedure for determining which type of left-turn lane is more cost effective on rural four-lane expressways in Nebraska was based on summing the annualized costs for Type A and Type B left-turn lanes and selecting the lower cost option. The costs used in the comparison were the operational, safety, construction, maintenance and operating costs associated with providing each type of left-turn lane. The determination of all costs relating to both left-turn lane options are outlined in this chapter.

Operational Costs

The operational costs were those associated with the delay and fuel consumption values for the Type A and B left-turn lanes. The hourly operational costs in delay and fuel consumption are computed using Equation 9.1.

$$(HC_{OP,\ A/B})_i = (DL_{A/B})(V_i/3600)(C_T) + 4(FC_{A/B})(C_F) \qquad \textbf{(9.1)}$$

where,

$(HC_{OP,\ A/B})_i$ = hourly operational costs on Type A or B approaches of a four-lane rural expressway in the i^{th} hour of the day, dollars per hour,
$DL_{A/B}$ = delay experienced on Type A or B approach from Equation 7.1, seconds per through vehicle,
$FC_{A/B}$ = fuel consumption on Type A or B approach link from Equation 7.2, gallons per 15 minutes,
V_i = approach volume in the i^{th} hour of the day, vehicles per hour,
C_T = unit value of travel time, dollars per hour, and
C_F = cost of fuel, dollars per gallon.

The approach volume in the i^{th} hour of the day used in Equation 9.1 was calculated by multiplying the current approach volume (expressway AADT divided by 2) by the appropriate average hourly percent value given in Table 8.6 for each of the 24 hours in the day.

The unit value of travel time, C_T, used to compute the operational costs was \$16.00 per hour. This value is the 1975 unit value of time established by AASHTO (*30*) updated to 2002 dollars in accordance with changes in the consumer price index. Dollar values of passenger car trips were based on average trip types, which assumed a vehicle occupancy of 1.56 persons per vehicle and a value of \$2.40 (1975 value) per person-hour. For trucks, travel-time savings were represented as market costs of \$7.00 (1975 value) per hour for single-unit trucks and \$8.00 (1975 value) per hour for combination trucks. These three values were summed and updated to 2002 dollars resulting in the \$16.00 unit value of time. The vehicle mix for rural expressways was estimated by the average composition of traffic at continuous counting stations on rural expressways in Nebraska (*23*). Vehicle mix percentages were 70 percent passenger cars, 22 percent single unit trucks, and 8 percent combination trucks.

The cost of fuel, C_F, used to compute the operational costs was \$1.35 per gallon. This was the average price of gasoline in September 2002 in Lincoln, NE.

The daily operational costs were computed by summing the hourly operational costs for the 24-hour period in the day as shown in Equation 9.2.

$$DC_{OP,\,A/B} = \sum_{i=1}^{24} (HC_{OP,\,A/B})_i \qquad \textbf{(9.2)}$$

where,

$DC_{OP,\,A/B}$ = daily operating costs for Type A and B approaches, dollars per day, and
$(HC_{OP,\,A/B})_i$ = hourly operational costs for Type A and B approaches in the i^{th} hour of the day, dollars per hour.

The annual operational costs were then computed by multiplying by 365 days in the year shown by Equation 9.3.

$$AC_{OP,\,A/B} = 365(DC_{OP,\,A/B}) \qquad \textbf{(9.3)}$$

where,

$AC_{OP,\,A/B}$ = annual operating costs, dollars per year, and
$DC_{OP,\,A/B}$ = daily operational costs on a four-lane rural expressway, dollars per day.

Accident Costs

Accident costs were those associated with rear-end and left-turn-leaving accident types for Type A and Type B left-turn lanes.

As explained in Chapter 8, the number of predicted accidents per year caused by the speed differentials of left-turning vehicles in the passing lane of rural expressways was calculated using Equation 8.9. The number of predicted left-turn-leaving accidents per year due to the lateral position of the left-turn storage lane (obstruction by opposing left-turn vehicle) was calculated using Equations 8.11 and 8.12.

According to the NDOR revised relative severity index figures from May 1999 (*31*), societal costs of Nebraska traffic accidents are given by the following values in Table 9.1.

Table 9.1 Relative Severity Index Figures for Accident Types (*31*)

Accident Type	Societal Cost
Fatal	$3,520,600
Disabling Injury	$295,600
Visible Injury	$62,500
Possible Injury	$32,600
Property Damage Only	$5,800

The societal cost of a rural rear-end multi-vehicle collision is $53,700 (*31*) but the value used in this research study was derived from the data shown in Table 9.3. The rear-end type of accident was used to represent the type of mishap that would occur if a left-turn vehicle decelerated within the high-speed lane of an expressway causing a following vehicle to collide with it. Another type of accident associated with a decelerating left-turn vehicle could be of the side-swipe type but this type of accident was not considered in this research project since a sideswipe crash could result from other causes than the deceleration of a left-turn vehicle. The NDOR rural expressway intersection accident database was searched to determine the total number of rear-end accidents from 1988 to 2000 and accident severity frequencies were compiled. Results are shown in Tables 9.2 and 9.3.

Table 9.2 Summary of Rear-End Accidents at Unsignalized Expressway Intersections in Nebraska, 1988-2000

Accident Type	Total Accidents	Fatal	Injury			Property Damage Only
			Disabling	Visible	Possible	
Rear End	13	0	1	0	4	8

Table 9.3 shows the values used to determine the societal cost of $36,338. This value was rounded to $36,300 for future calculations.

Table 9.3 Determination of Societal Cost Attributed to Rear-End Accidents on Unsignalized Rural Expressway Intersections in Nebraska

Accident Type	Portion of Accident Type with respect to Total Accidents[1]	Nebraska Societal Cost, Dollars[2]	Societal Cost for Left-Turn-Leaving Accidents by Accident Type, Dollars
Fatal	0/13	3,520,600	0
Disabling Injury	1/13	295,600	22738
Visible Injury	0/13	62,500	0
Possible Injury	4/13	32,600	10031
Property Damage Only	8/13	5,800	3569
			Sum = 36,338 ⇒ 36,300

[1] Values are from Table 9.2
[2] Reference (*31*)

The societal cost of a rural left-turn-leaving multi-vehicle collision is $94,700 (*31*) but the value used in this research project was derived from the accident data gathered in Table 8.8 which reflects unsignalized intersections on rural expressways in Nebraska. Table 9.4 shows the values used to determine the societal cost of $95,488. This value was rounded to $95,500 for future calculations.

Table 9.4 Determination of Societal Cost Attributed to Left-Turn-Leaving Accidents on Unsignalized Rural Expressway Intersections in Nebraska

Accident Type	Portion of Accident Type with respect to Total Accidents[1]	Nebraska Societal Cost, Dollars[2]	Societal Cost for Left-Turn-Leaving Accidents by Accident Type, Dollars
Fatal	3/207 = 0.0145	3,520,600	51023
Disabling Injury	14/207 = 0.0676	295,600	19992
Visible Injury	51/207 = 0.2464	62,500	15399
Possible Injury	40/207 = 0.1932	32,600	6300
Property Damage Only	99/207 = 0.4783	5,800	2774
			Sum = 95,488 ⇒ 95,500

[1] Values are from Table 8.11
[2] Reference (*31*)

The annual accident costs predicted for Type A and B left-turn lanes on 55-mph and 65-mph posted speed expressways with 15:1 tapers were computed using Equations 9.4 through 9.7.

$$AC_{ACC,\,55A} = \$36{,}300\ ACC_{R\text{-}E,\,55A} + \$95{,}500 ACC_{LT\text{-}T\text{-}LV,\,55A} \qquad \textbf{(9.4)}$$

$$AC_{ACC,\,55B} = \$36{,}300\ ACC_{R\text{-}E,\,55B} + \$95{,}500 ACC_{LT\text{-}T\text{-}LV,\,55B} \qquad \textbf{(9.5)}$$

$$AC_{ACC,\,65A} = \$36{,}300\ ACC_{R\text{-}E,\,65A} + \$95{,}500 ACC_{LT\text{-}T\text{-}LV,\,65A} \qquad \textbf{(9.6)}$$

$$AC_{ACC,\,65B} = \$36{,}300\ ACC_{R\text{-}E,\,65B} + \$95{,}500 ACC_{LT\text{-}T\text{-}LV,\,65B} \qquad \textbf{(9.7)}$$

where,

$AC_{ACC,\,55A}$ = annual accident cost savings on 55A left-turn lanes with 15:1 tapers,
$ACC_{R\text{-}E,\,55A}$ = rear-end accident cost savings on 55A left-turn lanes with 15:1 tapers,
$ACC_{LT\text{-}T\text{-}LV,\,55A}$ = left-turn-leaving accident cost savings on 55A left-turn lanes with 15:1 tapers,

$AC_{ACC,\,55B}$ = annual accident cost savings on 55B left-turn lanes with 15:1 tapers,
$ACC_{R\text{-}E,\,55B}$ = rear-end accident cost savings on 55B left-turn lanes with 15:1 tapers,
$ACC_{LT\text{-}T\text{-}LV,\,55B}$ = left-turn-leaving accident cost savings on 55B left-turn lanes with 15:1 tapers,

$AC_{ACC,\,65A}$ = annual accident cost savings on 65A left-turn lanes with 15:1 tapers,
$ACC_{R\text{-}E,\,65A}$ = rear-end accident cost savings on 65A left-turn lanes with 15:1 tapers,
$ACC_{LT\text{-}T\text{-}LV,\,65A}$ = left-turn-leaving accident cost savings on 65A left-turn lanes with 15:1 tapers, and

$AC_{ACC,\,65B}$ = annual accident cost savings on 65B left-turn lanes with 15:1 tapers,
$ACC_{R\text{-}E,\,65B}$ = rear-end accident cost savings on 65B left-turn lanes with 15:1 tapers, and
$ACC_{LT\text{-}T\text{-}LV,\,65B}$ = left-turn-leaving accident cost savings on 65B left-turn lanes with 15:1 tapers.

Table 9.5 shows a summary of the equations and constant values used to determine the cost differences of Type A and B left-turn lane types that are used in the procedure presented in Chapter 10.

Table 9.5 Summary of Equations and Constant Values of Annual Operational and Accident Costs

AC_{OP} = Annual Operational Costs ***(Applicable to Type A and B Left-Turns):***	**Constants Used in Chapter 10 Procedure**
$365\left(\sum_{i=1}^{24}\left[\left(\frac{V_i}{3600}\right)(C_T)e^{[-0.00161EL+0.0170LT+0.0740D]}+4(C_F)e^{[-0.000546EL+0.00550LT0.0120T+0.000918V]}\right]\right)$	
where,	
V_i = approach volume in the i^{th} hour of the day, vehicles per hour (=(AADT ÷ 2)(average hourly percent of traffic))	
C_T = unit value of travel time, dollars per hour,	\$16.00
EL = equivalent storage length (parallel storage length plus tapered lane length ≥ 9 feet wide),	
LT = approach left-turn volume, percent of AADT on expressway,	
D = approach density, total approach volume ÷ expected posted speed, vehicles per mile,	V÷55, V÷65
C_F = cost of fuel per gallon, and	\$1.35
T = percent of trucks, percent of AADT on expressway approach.	
$ACC_{R\text{-}E}$ = Annual Accident Costs Due to Speed Differential ***(Applicable to Type A and B Left-Turns):***	**Constants Used in Chapter 10 Procedure**
$\dfrac{\$36{,}300[P_D I_D + P_N I_N]\left[P_{LT}P_{PL}ADT_{EXPWY}\dfrac{L_{DEC}}{5280(365)}\right]}{2(10^8)}$	
where,	
P_D = portion of daytime traffic	0.73
I_D = daytime accident involvement rate, accidents per 100 million vehicle miles	$55A_D$,$55B_D$,$65A_D$,$65B_D$
P_N = portion of nighttime traffic,	0.27
I_N = nighttime accident involvement rate, accidents per 100 million vehicle miles,	$55A_N$,$55B_N$,$65A_N$,$65B_N$
P_{LT} = portion of left-turning vehicles on approach,	
P_{PL} = portion of total approach traffic in passing lane,	55 mph = 0.38, 65 mph = 0.22
$AADT_{EXPWY}$ = annual average daily traffic, vehicles per day, and	
L_{DEC} = left-turn deceleration distance, ft	450
$ACC_{LT\text{-}T\text{-}LV}$ = Annual Accident Costs Due to Removal of Sight Obstruction ***(Applicable to Type A and B Left-Turns):***	**Constants Used in Chapter 10 Procedure**
$\$95{,}500(A_R)(P_{LT\text{-}T\text{-}LV})\left[0.6503\left(\frac{ADT_{EXPWY(\ )}}{1000}\right)^{0.2925}\left(\frac{ADT_{MINOR}}{1000}\right)^{0.7911}\right]$	
where,	
A_R = accident reduction factor for using an offset left-turn lane	0.50 (Type A), 1.00 (Type B)
$P_{LT\text{-}T\text{-}LV}$ = portion of total accidents attributable to left-turn-leaving type	0.061
$AADT_{EXPWY}$ = annual average daily traffic, vehicles per day, and	
$AADT_{MINOR}$ = minor (cross) road traffic volume, vehicles per day.	

Left-Turn Lane Construction Costs

The construction costs of left-turn lanes were estimated from average unit prices for the various elements and tasks associated with left-turn lane construction on typical rural expressway projects in Nebraska provided by the Plans, Specifications and Estimates Section of NDOR. The primary left-turn lane construction material on rural expressways is concrete pavement. However, some left-turn bays are constructed using asphalt surfacing. Left-turn lanes are also normally built along with the construction of the four-lane expressway but there may also be situations where left-turn lanes are added to an existing expressway segment and construction costs may need to reflect a small-scale retrofit project circumstance. Intersection lighting is also a major construction cost factor. Requests to provide roadway lighting at rural intersections are studied individually by the Lighting Unit in the Roadway Design Division at the NDOR. If results indicate that prevailing conditions satisfy the requirements of one of the warrants shown in Table 9.6, a lighting system will be constructed subject to priority scheduling and availability of funds.

Table 9.6 Warrants for Rural Intersection Lighting on NDOR State Highway System (Outside of Corporate Limits)

Case Number	Case Warrant	Warrant Definition
I.	Accident History	Yearly nighttime accidents > 1/3 yearly daytime accidents **and** average yearly nighttime accidents > 3
II.	Cost Effectiveness	Cost effective analysis limited to certain types of intersections, based on FHWA/TRB/NDOR research project
III.	AADT/ Topography/ Geometrics	Intersection AADT > 2500 vpd and at least two of the following: • Complex or unusual geometrics, • Raised medians, • Inadequate sight distance, • Frequent pedestrian traffic, • High percentage of turning movements, • Confusing background lighting, and • Other adverse geometric, topographic or operational conditions.
IV.	Local Responsibility	Case I, II, III are not met but local governing authority finds sufficient benefit to pay for 50% of lighting installation and 100% of operation and maintenance costs of a lighting system.

To adequately represent all possible scenarios, costs have been determined for the various left-turn lane lengths representing Type A and B configurations using large and small project costs of concrete and asphalt surfacing types assuming that intersection lighting costs are included. Table 9.7 shows the cost of left-turn lane construction as a part of a large expressway project including lighting of the intersection. Table 9.8 shows the cost of left-turn lane construction as a part of a stand-alone retrofit expressway project including lighting of the intersection. Table 9.9 shows equations that can be used to determine the cost of a left-turn lane design option based on the equivalent storage length of the left-turn lane.

Table 9.7 Cost of Left-Turn Lane Construction as a Part of a Large Expressway Project (Includes Lighting of the Intersection)

Left-Turn Type	Length Of Parallel Storage Lane (Equivalent Storage Length), Feet	Variable Cost, Concrete (Part of Large Project), Dollars	Variable Cost, Asphalt (Part of Large Project), Dollars	Mobil-Ization (included in Variable Cost), Dollars	Preliminary, Construction Engineering, Contingencies (18.4% of Construction Costs*), Dollars		Total Cost, Dollars	
					Conc	Asph	Conc	Asph
A	70 (370)	81888	66655	3700	15068	12265	96956	78920
A	250 (550)	100290	77108	3700	18454	14188	118744	91296
$B_{residential}$	0 (90)	53717	48891	2000	9884	8996	63601	57887
$B_{county\ road}$	60 (150)	56105	50388	2000	10324	9272	66429	59660
$B_{county\ road}$	140 (230)	59289	51950	2000	10910	9559	70199	61509
$B_{major\ road}$	320 (410)	66463	55469	2000	12230	10207	78693	65676
$B_{major\ road,\ level}$	670 (760)	80374	62292	2000	14789	11462	95163	73754
$B_{major\ road,\ 4\%\ dn}$	750 (840)	83567	63860	2000	15377	11751	98944	75611

* Preliminary Engineering = 4.4% of Construction Costs, Construction Engineering = 9.0% of Construction Costs Contingencies = 5.0% of Construction Costs

Table 9.8 Cost of Left-Turn Lane Construction as a Stand-Alone Retrofit Project (Includes Lighting of the Intersection)

Left-Turn Type	Length Of Parallel Storage Lane (Equivalent Storage Length), Feet	Variable Cost, Concrete (Part of Large Project), Dollars	Variable Cost, Asphalt (Part of Large Project), Dollars	Mobil-Ization (included in Variable Cost), Dollars	Preliminary, Construction Engineering, Contingencies (18.4% of Construction Costs*), Dollars		Total Cost, Dollars	
					Conc	Asph	Conc	Asph
A	70 (370)	124239	104998	16300	22860	19320	147099	124318
A	250 (550)	146339	117111	16300	26927	21549	173266	138660
$B_{residential}$	0 (90)	84830	74656	8400	15609	13737	100439	88393
$B_{county\ road}$	60 (150)	89553	76629	8400	16478	14100	106031	90729
$B_{county\ road}$	140 (230)	95871	79269	8400	17641	14586	113512	93855
$B_{major\ road}$	320 (410)	110041	84589	8400	20248	15565	130289	100154
$B_{major\ road,\ level}$	670 (760)	137554	96683	8400	25310	17790	162864	114473
$B_{major\ road,\ 4\%\ dn}$	750 (840)	143870	99321	8400	26473	18276	170343	117597

* Preliminary Engineering = 4.4% of Construction Costs, Construction Engineering = 9.0% of Construction Costs Contingencies = 5.0% of Construction Costs

Table 9.9 Equations for Cost (Dollars) of Left-Turn Lane with Lighting Based on Equivalent Length of Left-Turn Lane

Type	Material	Large Project/Retrofit	Equation	Equation Number
A	Concrete	Large Project	$121.040EL + 52170$	**9.8**
A	Asphalt	Large Project	$168.756EL + 53480$	**9.9**
B	Concrete	Large Project	$47.116EL + 59364$	**9.10**
B	Asphalt	Large Project	$23.357EL + 56029$	**9.11**
A	Concrete	Retrofit	$145.370EL + 93311$	**9.12**
A	Asphalt	Retrofit	$79.678EL + 94837$	**9.13**
B	Concrete	Retrofit	$93.181EL + 92065$	**9.14**
B	Asphalt	Retrofit	$38.934EL + 84774$	**9.15**

Note: EL = Equivalent length of left-turn lane

The variable EL in Equations 9.8 to 9.15 represents the equivalent storage length of the left-turn lane. The equivalent storage length is defined as the longitudinal length of a left-turn lane which can store vehicles safely beyond the edge of the adjacent through traffic lane. The distance can be calculated by determining the longitudinal distance along the tapered segment which is greater than 9 ft wide plus the length of parallel lane required for storage of left-turn vehicles in the 1996 NDOR Roadway Design Manual discussed in Chapter 2. Figure 9.1 shows a diagram of the equivalent storage length on a Type A and Type B left-turn lane configuration. The longitudinal distance along the tapered segment greater than 9 ft for Type A and B left-turn lanes is shown in Table 9.10.

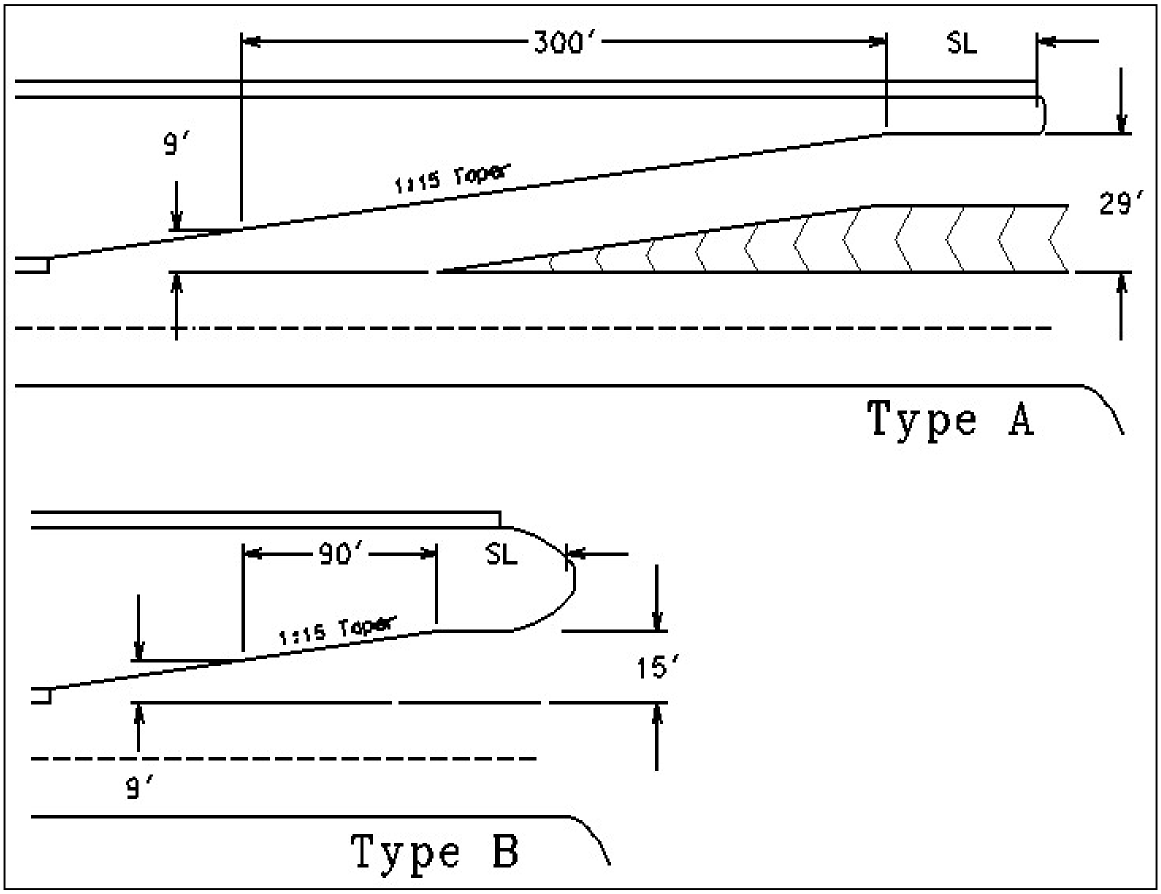


Figure 9.1 Equivalent Storage Lengths

Table 9.10 Length of Taper Available for Storage of Left-Turn Vehicles (EL, Equivalent Storage Length) on Type A and B Left-Turn Types According to the 1996 Roadway Design Manual (*1*)

Left-Turn Lane Type	Taper Rate Longitudinal to Transverse, ft	Length of Taper Available for Storage of Left-Turn Vehicles, ft
A	15:1	300
B	15:1	90

Tables 9.11 through 9.13 show similar costs and equations **without** intersection lighting costs included.

It was assumed that no utility costs would be incurred and no additional right-of-way for left-turn lanes would be required since both left-turn lane types would be constructed within the right-of-way purchased for the construction of the full width of the expressway.

Table 9.11 Cost of Left-Turn Lane Construction as a Part of a Large Expressway Project (No Lighting of the Intersection Included)

Left-Turn Type	Length Of Parallel Storage Lane (Equivalent Storage Length), Feet	Variable Cost, Concrete (Part of Large Project), Dollars	Variable Cost, Asphalt (Part of Large Project), Dollars	Mobil-Ization (included in Variable Cost), Dollars	Preliminary, Construction Engineering, Contingencies (18.4% of Construction Costs*), Dollars		Total Cost, Dollars	
					Conc	Asph	Conc	Asph
A	70 (370)	41501	26268	3700	7637	4834	49138	31102
A	250 (550)	59903	36721	3700	11023	6757	70926	43478
$B_{residential}$	0 (90)	13330	8504	2000	2453	1565	15783	10069
$B_{county\ road}$	60 (150)	15718	10001	2000	2893	1841	18611	11842
$B_{county\ road}$	140 (230)	18902	11563	2000	3478	2128	22380	13691
$B_{major\ road}$	320 (410)	26076	15082	2000	4798	2776	30874	17858
$B_{major\ road,\ level}$	670 (760)	39987	21905	2000	7358	4031	47345	25936
$B_{major\ road,\ 4\%\ dn}$	750 (840)	43180	23473	2000	7946	4320	51126	27793

* Preliminary Engineering = 4.4% of Construction Costs, Construction Engineering = 9.0% of Construction Costs
Contingencies = 5.0% of Construction Costs

Table 9.12 Cost of Left-Turn Lane Construction as a Stand-Alone Retrofit Project (No Lighting of the Intersection Included)

Left-Turn Type	Length Of Parallel Storage Lane (Equivalent Storage Length), Feet	Variable Cost, Concrete (Part of Large Project), Dollars	Variable Cost, Asphalt (Part of Large Project), Dollars	Mobil-Ization (included in Variable Cost), Dollars	Preliminary, Construction Engineering, Contingencies (18.4% of Construction Costs*), Dollars		Total Cost, Dollars	
					Conc	Asph	Conc	Asph
A	70 (370)	80484	61243	16300	14810	11269	95294	72512
A	250 (550)	102584	73356	16300	18876	13498	121460	86854
$B_{residential}$	0 (90)	41075	30901	8400	7558	5686	48633	36587
$B_{county\ road}$	60 (150)	45798	32874	8400	8427	6049	54225	38923
$B_{county\ road}$	140 (230)	52116	35514	8400	9590	6535	61706	42049
$B_{major\ road}$	320 (410)	66286	40834	8400	12197	7514	78483	48348
$B_{major\ road,\ level}$	670 (760)	93799	52928	8400	17260	9739	111059	62667
$B_{major\ road,\ 4\%\ dn}$	750 (840)	100115	55566	8400	18422	10225	118537	65791

* Preliminary Engineering = 4.4% of Construction Costs, Construction Engineering = 9.0% of Construction Costs
Contingencies = 5.0% of Construction Costs

Table 9.13 Equations for Cost (Dollars) of Left-Turn Lane without Lighting Based on Equivalent Length of Left-Turn Lane

Type	Material	Large Project/Retrofit	Equation	Equation Number
A	Concrete	Large Project	$121.040EL + 4352$	**9.16**
A	Asphalt	Large Project	$168.756EL + 5662$	**9.17**
B	Concrete	Large Project	$47.116EL + 11545$	**9.18**
B	Asphalt	Large Project	$23.357EL + 8211$	**9.19**
A	Concrete	Retrofit	$145.370EL + 41508$	**9.20**
A	Asphalt	Retrofit	$79.678EL + 43031$	**9.21**
B	Concrete	Retrofit	$93.182EL + 40259$	**9.22**
B	Asphalt	Retrofit	$38.934EL + 32968$	**9.23**

Note: EL = Equivalent Storage Length of left-turn lane.

Left-Turn Lane Maintenance Costs

Costs to maintain surfacing on expressways were estimated from recorded maintenance activities in the seven-year period from 1995 through 2001 along the five one-mile segments of expressway where data was collected for this project. Average annual costs for spot patching, milling and machine patching were developed in terms of dollars per square foot of surfacing per year to reflect a maintenance cost based on surface area. Annual lighting maintenance costs were averaged for two of the five study locations which had lighting. Table 9.14 shows the resulting annual maintenance costs.

Table 9.14 Annual Maintenance Costs for Expressway Intersections with Left-Turn Lanes

Maintenance Cost Item	Annual Cost, dollars per year
Surfacing Maintenance	\$0.00126 per square foot of surfacing
Lighting Maintenance	\$400 per intersection installation

Left-Turn Lane Operating Costs

Operating costs were considered to be those associated with energy costs for intersection lighting. The total energy cost depends upon the size and type of luminaire and the utility company serving the units. Costs were tabulated for ten 200-watt high-pressure sodium luminaires (standard lighting installation for an expressway intersection) which the NDOR Lighting Unit estimated to be \$6 per unit per month. The annual operating cost is shown in Table 9.15.

Table 9.15 Annual Operating Costs for Expressway Intersections with Left-Turn Lanes and Lighting

Operating Cost Item	Annual Cost, dollars per year
Lighting Energy Costs	\$720

Summary of Costs for Left-Turn Lanes on Rural Expressways in Nebraska
Operational, accident, construction, maintenance and operating costs of Type A and B left-turn lanes that were determined using the equations developed in this chapter were used to arrive at a procedure which assesses which type of left-turn lane is least costly. An application of this procedure is detailed in an example included in Chapter 10.

Chapter 10
APPLICATION

This chapter illustrates the use of the equations developed in the research project to establish which left-turn design is most cost effective for a particular situation. Given known information about current (beginning year of operation) traffic volumes and expected posted speed, the following procedure determines if a Type A or B left-turn lane design is the most cost effective based on a comparison of the annualized costs. The left-turn lane type resulting in the lowest cost is the most cost-effective left-turn lane type to construct. Type A and B left-turn lanes are assumed to have 15:1 tapers. Equation 10.1 shows the various costs considered in the procedure.

$$\begin{gathered}\text{Annualized Costs of Type A or Type B Left-Turn Lanes} = \qquad \textbf{(10.1)}\\ (\text{Annualized Operational Costs})_{\text{Type A or B}} + (\text{Annualized Predicted Accident Costs})_{\text{Type A or B}} +\\ (\text{Annualized Construction Costs})_{\text{Type A or B}} + (\text{Annualized Maintenance Costs})_{\text{Type A or B}} +\\ (\text{Annualized Operating Costs})_{\text{Type A or B}}\end{gathered}$$

Input Values
The following input values must be known or estimated to perform the procedure outlined in this chapter.

1. **Current expressway annual average daily traffic: vehicles per day,**
   This value should represent the annual average daily traffic in both directions on the expressway at the time the newly constructed expressway project is open to traffic.
2. **Current crossroad annual average daily traffic: vehicles per day,**
   This value should represent the annual average daily traffic in both directions on the crossroad at the time the newly constructed expressway project is open to traffic.
3. **Category of crossroad: major intersection, standard county road, or residential drive:**
   According to the 1999 revised Type B median break criteria, the Type B left-turn lane geometrics are partially based on deceleration length as described in Chapter 2. The initial speed used to calculate the deceleration to be accommodated by the left-turn lane design is dependent on the designation of the intersection as a major intersection, standard county road intersection, or a residential drive. Unfortunately, there is currently no criteria by which to judge what crossroad volume designates a major intersection from a standard county road intersection. A crossroad with 750 or less AADT shall be considered a standard county road intersection for purposes of this benefit/cost analysis. This value represents the highest volume shown in the 2002 Minimum Design Standards for a rural road (*32, p. 21*).
4. **Percent of heavy trucks on approach, percent:**
   This value should represent the percent of heavy trucks within the total traffic on the approach lanes. The value of AADT for the expressway should be divided by 2 to get the volume of traffic in one direction for the total approach traffic before determining the percentage of heavy trucks.
5. **Percent of left-turns on approach, percent:**
   This value should represent the percent of left-turn vehicles within the total traffic on the approach lanes. The value of AADT for the expressway should be divided by 2 to get the

volume of traffic in one direction for the total approach traffic before determining the percentage of left-turn vehicles.

6. **Approach grade, ft/ft:**
   The approach grade of the left-turn lane is required to determine the length of the Type B left-turn lane designated for deceleration. See Table 10.1 below.
7. **Equivalent length of storage lane, ft:**
   The equivalent storage length is defined as the area within a left-turn lane which can store vehicles safely beyond the edge of the adjacent through traffic lane. The distance can be calculated by determining the longitudinal distance along the tapered segment greater than 9 ft wide plus the length of parallel lane required for storage of left-turn vehicles calculated using the 1996 NDOR Roadway Design Manual method discussed in Chapter 2 plus additional length required for deceleration (applies to Type B only) or a minimum storage lane length. This method requires the design hourly volume of the left-turn vehicles.
   Type A: The equivalent storage length for a Type A left-turn lane is equal to 300 ft within the taper plus at least a minimum storage length of 70 ft according to Figure 1.1.
   Type B: The equivalent storage length for a Type B left-turn lane is equal to 90 ft within the taper plus additional distance for deceleration if the crossroad is a major roadway (greater than 750 ADT) or a standard county road (750 ADT or less) plus a minimum parallel storage lane length.

Table 10.1 Determination of Deceleration Length for Type B Left-Turn Lane Based on 1999 Revised Type B Median Break Criteria (*2*)

Crossroad Designation	Crossroad ADT, vpd (*32*)	Initial Speed Assumed for Braking, mph (*2*)	Deceleration Length, ft (*2*)	Taper Distance used for Braking, ft (*2*)	Net Deceleration Length, ft (*2*)
Major Roadway	> 750	58	$58^2/30\ (0.27 \pm G^*)$	45	$(58^2/30\ (0.27 \pm G^*)) – 45$
Standard County Road	≤ 750	44	$44^2/30\ (0.27 \pm G^*)$	45	$(44^2/30\ (0.27 \pm G^*)) - 45$
Residential Drive	Not applicable	Not applicable	45	45	0

* G = profile grade in ft/ft

8. **Expected posted speed of expressway, mph:**
   Expressway posted speeds currently observed in the state of Nebraska are 55 mph and 65 mph. The Traffic Engineering Division at NDOR should be consulted to establish the value to be used.
9. **Type of project, large or retrofit:**
   The type of project under which the left-turn lane will be built will either be a large, major construction, expressway project or a small, retrofit project.

10. Construction Material Type, Concrete or Asphalt:

The type of construction material will be either concrete or asphalt for Type A or B left-turn lane types. The Materials and Research Division at NDOR should be consulted to determine the appropriate material type.

11. Intersection Lighting Requirement, Lighting/No Lighting,

The Lighting Engineer in the Roadway Design Division at NDOR should be consulted to determine if the intersection under consideration will require lighting.

Example

Table 10.2 shows example values that will be used in the application of the equations defined in this research project.

Table 10.2 Input Items for Determination of Most Cost-Effective Left-Turn Lane Type

Input Item	Input Item Value		Source of Input
Current Expressway AADT, vpd	12,000		Planning and Project Development Division and/or Traffic Division
Current Crossroad AADT, vph	1100		Planning and Project Development Division and/or Traffic Division
Crossroad Designation	Major Roadway		2002 Minimum Design Standards
Percent Heavy Trucks on Approach, %	12		Planning and Project Development Division and/or Traffic Division
Percent Left-Turn Vehicles on Approach, %	8		Planning and Project Development Division and/or Traffic Division
Approach Grade	**Type A**	**Type B**	Profile grade of expressway
	Not Applicable	G = 0.02 ft/ft	
Equivalent Length of Left-Turn Storage Lane, ft	300 + 125* = 425	90 + 387* + 125** = 602	Calculated value determined by left-turn lane type and required storage lane length
Approximate Area of Left-Turn Lane, sq ft	9933	7849	Taper + Deceleration (B only) + Storage Length
Expected Posted Speed Limit, mph	65 (rural)		Planning and Project Development Division and/or Traffic Division
Type of Project	Large Expressway Project		Planning and Project Development Division and/or Traffic Division
Material Type	Concrete for Type A, B		Materials and Research Division
Intersection Lighting	No		Lighting Engineer in Roadway Design Division

* The DHV of the expressway was determined by using the NDOR formula for "Other Rural Highways" category of Reference *23* which is approximately DHV = (0.10)(AADT) resulting in a DHV of 1200 for this example. Since left-turns represent 8% of the approach volume, 97 left-turns would occur in the design hour. This value was multiplied by a 60-second time interval (recommendation in NDOR Roadway Design Manual for rural conditions) and divided by 3600 to determine the average number of left-turn vehicles that would arrive per 60-second interval and was calculated to be 1.62. According to Table 2.2, the parallel storage length should be 125 ft. More detailed left-turn lane volume information may be available from the Planning and Project Development Division and/or Traffic Division.

**The deceleration length for the Type B left-turn lane is calculated using an initial speed of 58 mph, an asphalt friction factor of 0.27 and a profile grade of +0.02 ft/ft.

Tables 10.3 and 10.4 give summaries of constants and assumed variables used for the annualization of costs to determine which left-turn type has the lowest annual cost.

Table 10.3 Summary of Constants Used in Operational and Safety Costs Equations

Constant Symbols used in Operational and Accident Cost Equations	Symbol Definition	Assumed Constant Values used in Operational and Accident Cost Equations
C_T	Unit value of travel time	$16.00
C_F	Cost of fuel per gallon	$1.35
P_D	Portion of daytime traffic	0.73
P_N	Portion of nighttime traffic	0.27
P_{PL}	Portion of total approach traffic in passing lane	55 mph posted speed = 0.38 65 mph posted speed = 0.22
L_{DEC}	Left-turn deceleration distance in through traffic lane	450 ft
A_R	Accident reduction factor for using an offset left-turn lane	0.50 for Type A 1.00 for Type B
$P_{LT\text{-}T\text{-}LV}$	Portion of total accidents attributable to left-turn-leaving type	0.061

Table 10.4 Summary of Variables Used in the Annualization of Operational, Safety, Construction, Maintenance, and Operating Costs

Annualization Factor	Assumed Annualization Factor Value
Traffic growth rate	2.5% per year
Project design life	20 years
Discount rate	4%
Salvage value	$0

Spreadsheet Software

Microsoft Excel spreadsheets containing all calculations necessary for a cost comparison of Type A and Type B left-turn lanes has been created for NDOR as part of this research project. Example values shown in Table 10.2 are input in the spreadsheet on the first sheet file and the output values are calculated and appear on the first sheet after all input values have been inserted. The first sheet file is shown in Figure 10.1 for a Type A left-turn lane using the example values. Detailed instructions for the use of the spreadsheets are given in Appendix B.

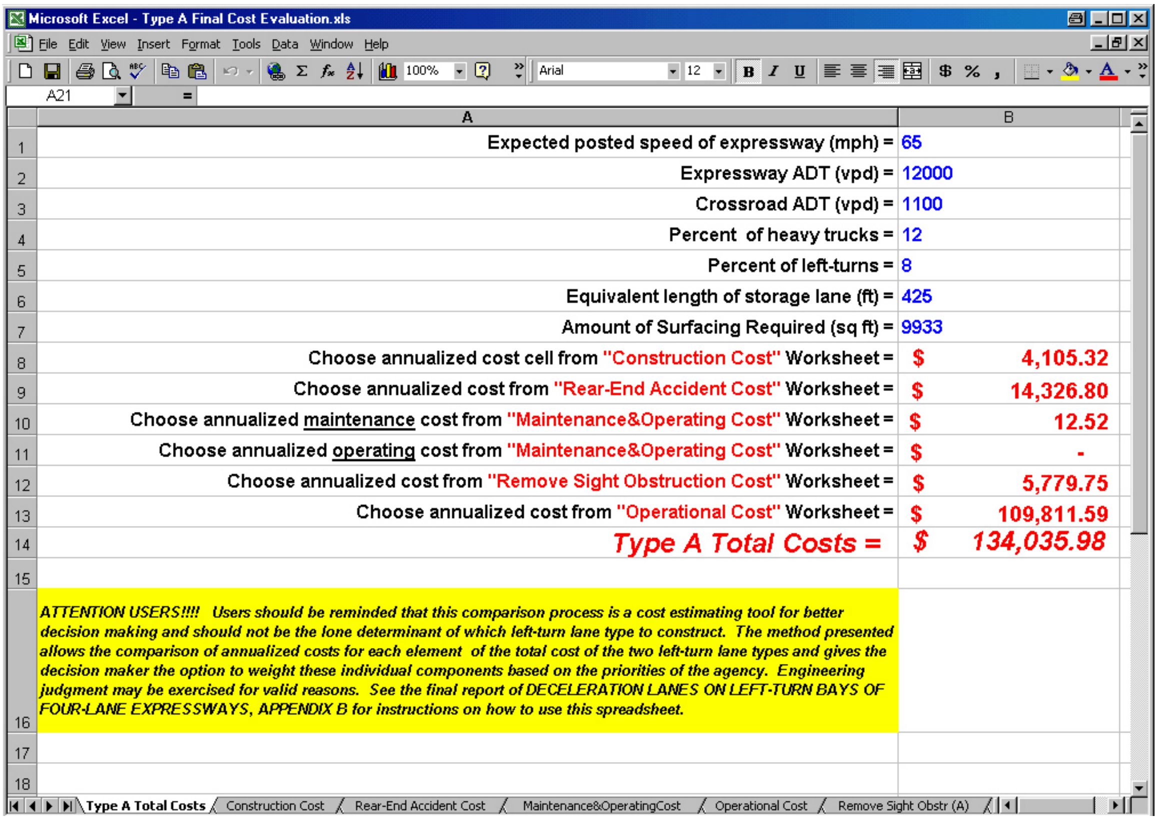


Figure 10.1 Input/Solution Page in Calculation Spreadsheet for Type A Left-Turn Lanes Using Example Values

Example Results

Table 10.5 shows the results of summing the construction, maintenance, operating, operational, and safety costs of both the Type A and Type B left-turn lanes given the example values.

Table 10.5 Results of the Comparison of Type A and B Costs for Example

Cost Item	Type A Annualized Cost, Dollars	Type B Annualized Cost, Dollars
Construction	4,105	2,936
Maintenance	13	10
Operating	0	0
Operational	109,812	96,870
Safety (Speed Differential)	14,327	3,100
Safety (Sight Distance Obstruction)	5,780	11,560
Total	**134,037**	**114,476**

The lowest cost option for the given situation is the Type B left-turn lane. Process users should be reminded that this comparison process is a cost-estimating tool for better decision making and should not be the lone determinant of which left-turn lane type to construct. The method presented in this research project allows the comparison of annualized costs for each element of the total cost of the two left-turn lane types and gives the decision maker the option to weight these individual components based on the priorities of the agency. Engineering judgment may be exercised for valid reasons. One such reason may be if this example situation resulted in having only one Type B left-turn lane amongst many other Type A configurations along an expressway segment and it is thought that allowing the construction of a single Type B configuration would confuse left-turning drivers.

Chapter 11
Recommendations for a Revised Offset Left-Turn Lane GEOMETRIC CONFIGURATION, TYPE AA

Research findings concerning operations and safety at left-turn lanes on rural expressways leads to the modification of geometric elements of Type A offset left-turn lanes to better fit the expectations of drivers and maximize the benefits that an offset left-turn lane affords. Due to the negative safety effects that are associated with the geometric design of the Type A offset left-turn lane evident at the study sites, it was necessary to propose a revised geometric configuration that would improve the safety effects of Type A left-turn lanes. The proposed design will henceforth be referred to as the Type AA left-turn design. The Type AA design features exhibit geometric modifications to provide the following positive left-turn features:

- Adequate offset that will assure minimum intersection sight distance for a worst-case situation for critical time gap (WB-62 combination truck on a 4 percent upgrade which is the steepest profile grade allowable on an expressway in Nebraska according to the 2002 Minimum Design Standards (*32*) turning left when opposed by a WB-62 combination truck),
- Feasible allocations for through-lane separator width, left-turn lane width, medial separator width, and offset to opposing left-turn lane meet or exceed the criteria listed in NCHRP 375 (*9*),
- Sufficient lengths that will accommodate the kinematic requirements of the left-turning and through vehicles according to the recommendations derived from previous research,
- Sufficient advance notice that a left-turn lane is available on the roadway ahead that will allow appropriate time for the driver to read the advance signs and perceive and react to the choice to make a left turn,
- Allowance for deceleration in gear for 3 seconds followed by comfortable braking completely within the left-turn lane (braking is assumed to begin where two-thirds of a 12-ft lane width (8 ft) is available (*3*)),
- Allowance for a 2.2 fps^2 (deceleration in gear) to 4.4 fps^2 (reasonable braking) deceleration rate (*4, p. 192*) for lateral movement while the left-turning vehicle is still within the limits of the through passing lane of the expressway to prevent the driver from developing a speed differential greater than 10 mph,
- Appropriate lane markings on the pavement at strategic locations to reinforce the driver's action of making a left-turn movement,
- Appropriate taper rate so through drivers do not mistake the left-turn lane for an additional through lane and left-turn drivers are not presented with a severe perspective angle causing them to decelerate abruptly in the passing lane,
- Appropriate painted lane width on the tapered section to allow for variation in entry angle of left-turning vehicles and minimize the wearing of paint stripes,
- Appropriate painted lane width on the parallel storage lane portion of the left-turn lane to assure that the driver centers the vehicle between the lane lines to obtain the necessary offset of the driver's eye from the corner of the opposing vehicle,

- Appropriate treatment of the medial separator to assure a contrasting, textured surface that reinforces the “traditional left-turn lane” appearance to a driver and considers maintenance and drainage issues common to narrow, turf medians,
- Appropriate surfacing material that will allow the best contrast between pavement markings and surfacing color,
- Provide sufficient storage to assure a minimal probability that the stacking of vehicles in the tapered section would obstruct the view of opposing left-turn vehicles, and
- Provide enough paved surface area to allow the wheels of a left-turning WB-62 combination truck to avoid the edge of surfacing by a distance of 2 ft.

The proposed Type AA design is shown in Figure 11.1. This design strives to provide all of the features listed above. The following numbered points apply to the items numbered in Figure 11.1. The item number is listed along with each item’s justification.

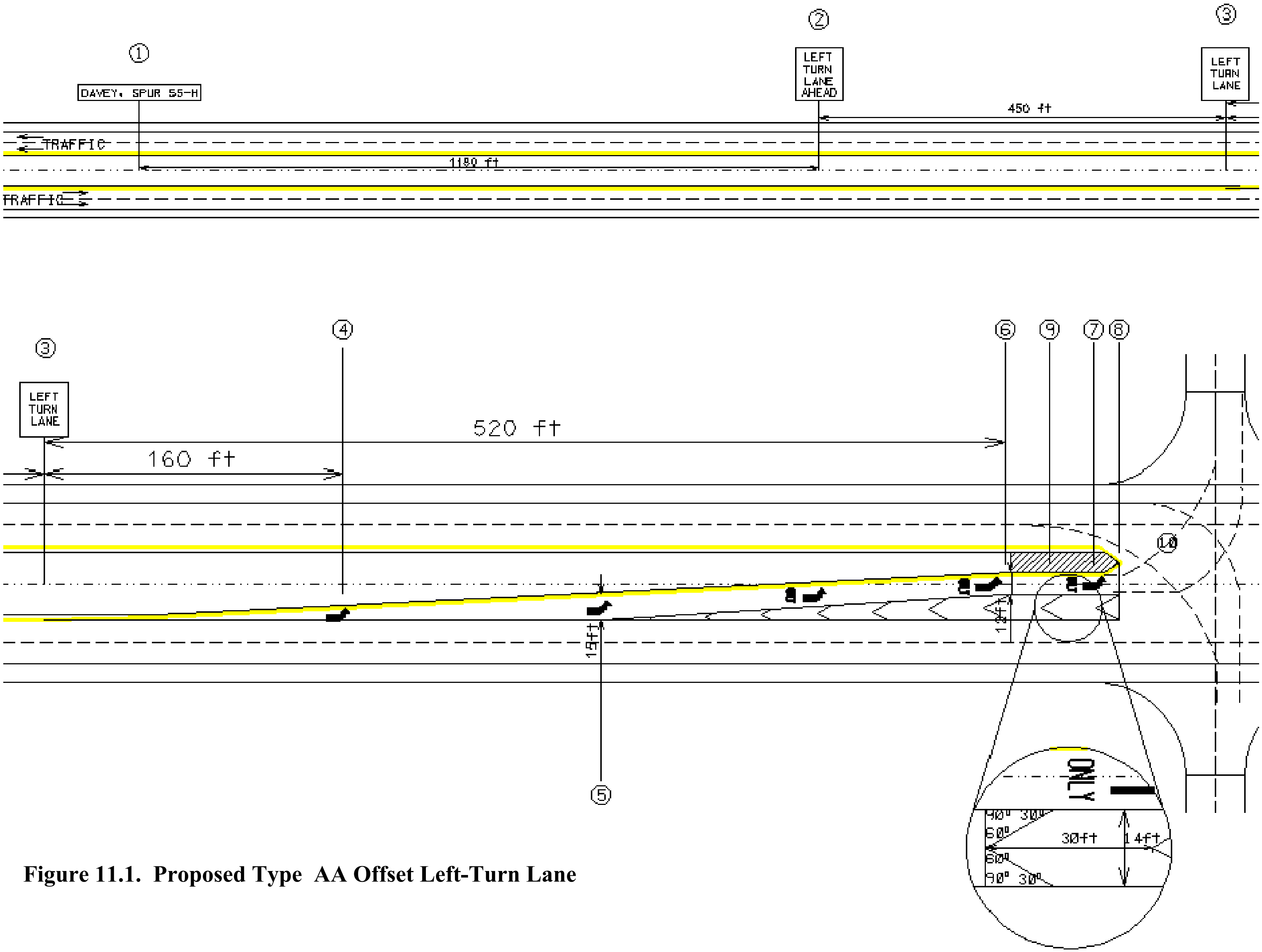


Figure 11.1. Proposed Type AA Offset Left-Turn Lane

1. **Advance guidance signing describing the highway/link/spur number and town name if applicable (MUTCD Code Number D3-2).** This sign should be placed at distance associated with the minimum decision sight distance for a speed/path/direction change on a rural road minus the distance from which the sign is legible to the driver. The 95th-percentile speed of vehicles in the passing lane 900-1200 ft in advance of the taper on the 55A and 55B sites was approximately 70 mph while the 95th-percentile speed at the same location of the 65A and 65B sites was about 75 mph. If the design speed reflects the 95th-percentile speed of 75 mph, the decision sight distance is 1180 ft for the worst-case of a 65-mph posted speed expressway, according to the decision site distance table in the 2001 Green Book (*4, p. 116*). Sign placement should be 850 ft from the expected path change which subtracts the sign legibility distance of 330 ft for 8-inch letters given a 40 ft per inch legibility requirement (*20, p. 2A-10*), 3 seconds of perception-reaction time and assuming that the legibility distance has to be greater than or equal to the perception-reaction distance for the 95th-percentile speed of 75 mph.
2. **Advance regulatory sign "Left Turn Lane Ahead" (MUTCD Code Number R3-20-30).** Letter height should be at least 8 inches. This sign should be placed 450 ft in advance of the taper. Evidence from all posted-speed/type left-turn options proved that no matter the left-turn lane configuration, drivers began their deceleration at a distance 450 ft in advance of the beginning of the taper. There is no expectation that the modified geometrics of the Type AA left-turn lane will change this behavior. This sign will reinforce the information presented in the guidance sign described in Item 1.
3. **Advance regulatory sign "Left Turn Lane" (MUTCD Code Number R3-20-30).** Letter height should be at least 8 inches. This sign should be placed adjacent to where the yellow painted taper line begins at the left edge of the passing lane. The surfacing type of the left shoulder is changed to asphalt at this location if the shoulder in advance of this point is constructed of concrete. The sign, taper line, and contrasting surfacing clearly delineate the presence of the left-turn lane and its beginning. The 3 ft shoulder is poured as a unit with the 12 ft passing lane in most cases of expressways with concrete driving lanes and shoulders. This may have a negative effect on the driver's perception of the available width of the left-turn lane. Changing the surfacing type at the edge of the passing lane should be a positive reinforcing stimulus for the driver to recognize the width of the left-turn lane.
4. **Beginning of 20:1 taper.** The 20:1 longitudinal-to-lateral taper for development of the left-turn lane also begins at this point. This taper is modified from the 15:1 recommended in the 1996 Roadway Design Manual on the Type A left-turn lane. The flatter taper was used to emulate the visual view of the left-turn lane that the driver sees when approaching a traditional left-turn lane similar to the configuration of the 65B study site. This shallow taper should influence drivers to enter the Type AA lane at a higher speed; thus reducing their deceleration in the passing lane which was found to be a problem with the Type A left-turn lane in the field studies.

   The horizontal viewing angle from the beginning to the end of the taper was calculated at three points for each design type. The three points were located in the center of the passing lane at distances of 50, 175 and 300 ft in advance of the taper.These angles are shown in Table 11.1. A negative angle indicates that the end of the taper is viewed as being to the left of the beginning of the taper. A positive angle indicates the taper end is viewed as being to the right of the taper beginning. A positive horizontal viewing angle at the 65A-Firth study site is shown in Figure 11.2. Perceptual views of the Type A and Type B left-turn lane configurations at the study sites are shown in Figure 11.3. The left-edge of the Type A left-turn lane looks as though it is built at a severe angle with respect to the through traffic lanes in the perspective view from a distance of 300 ft in advance of the taper. A similar photograph at the 65B site shows the Type B left-turn lane appearing to be at a shallower angle with respect to the alignment of the through lanes. As the driver approaches the intersection, the perceived angle of the Type A and B left-turn tapers increase with the perceived angle of the Type B design always appearing to be shallower than that of the Type A design. The perceived angle of the Type A design does not become negative until a distance of 50 ft in advance of the taper but, at this point, drivers were observed to have already reduced their speed in the passing lane. Apparently, the steeper taper angle perceived by drivers caused them to slow in anticipation of having to enter the left-turn lane at a high angle of departure.

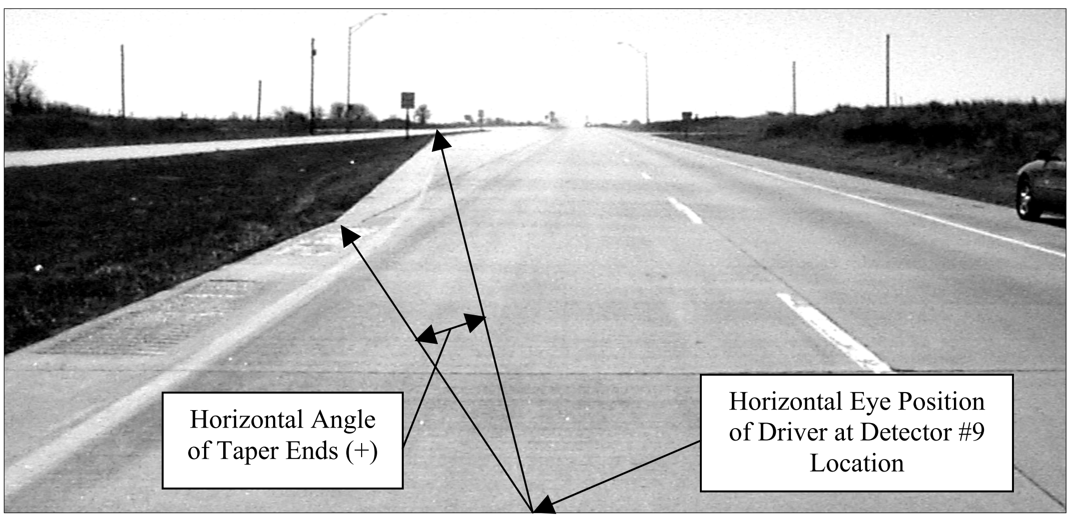
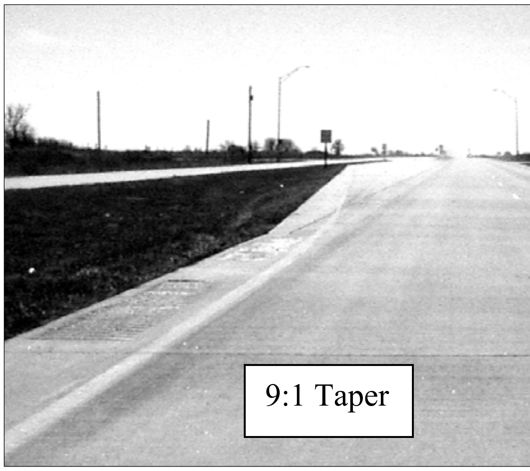


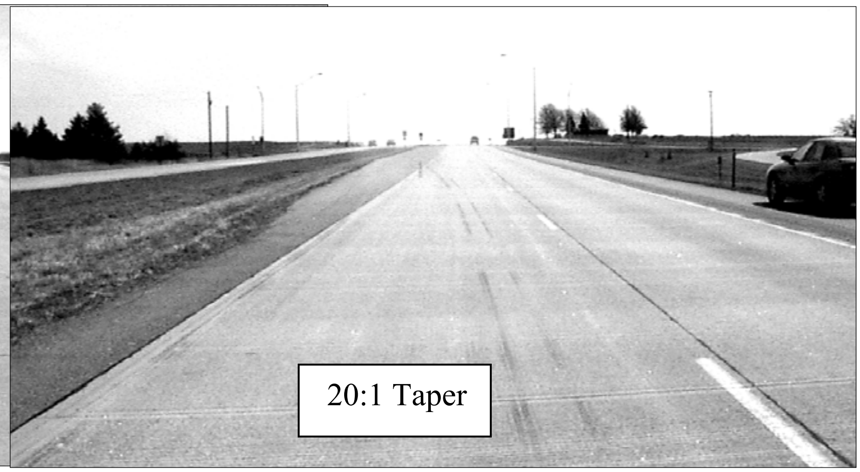
Figure 11.2 Positive Horizontal Viewing Angle from Beginning to End of Left-Turn Taper

Table 11.1 Horizontal Viewing Angle from Beginning to End of Left-Turn Taper

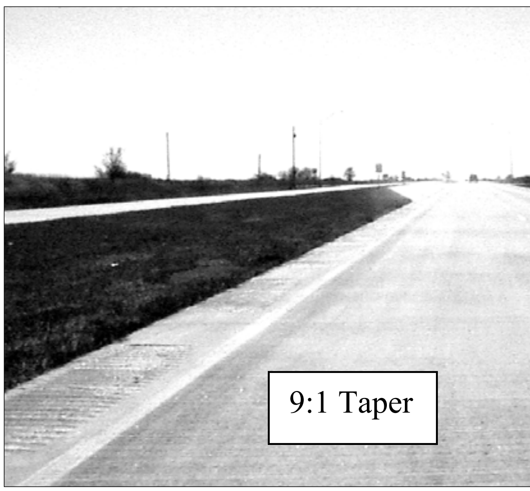
Design Type	Taper Rate (Longitudinal to Lateral, ft:ft)	Horizontal Viewing Angle, Degrees — Distance in advance of taper: 50 ft	175 ft	300 ft
Site 1 - 55A	7:1[1]	-1.0335	-3.3264	-2.8346
A (Design Manual)	15:1	2.7152	-1.3202	-1.5805
B (Design Manual)	15:1	2.4760	-1.0416	-1.1448
Site 5 – 65B	20:1[2]	3.4092	-0.5677	-0.8587
AA (Proposed)	20:1	3.6296	-0.6725	-1.0890

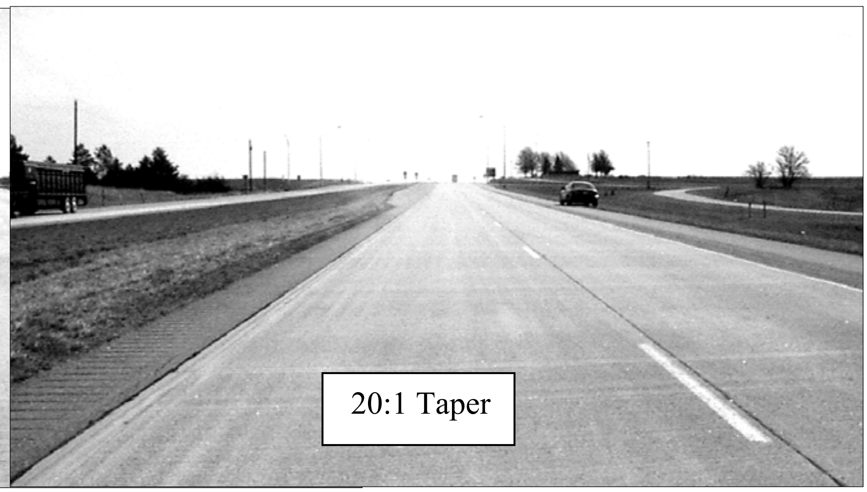
[1] Steepest study site taper rate
[2] Shallowest study site taper rate



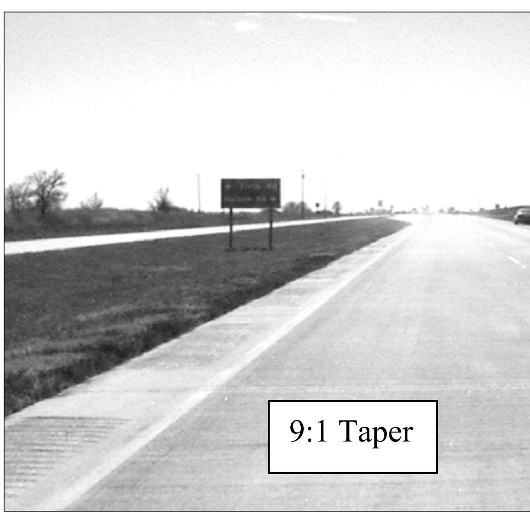


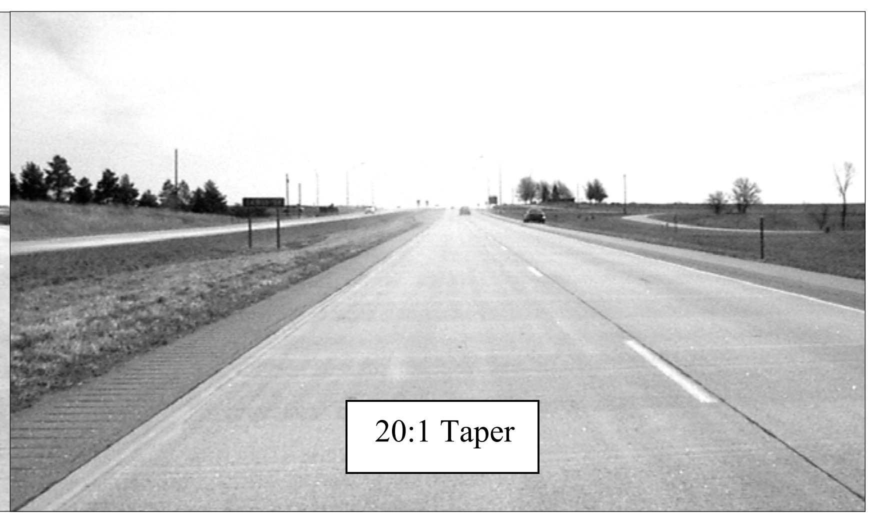
Photograph 3, 65A Firth, 50 ft in Advance of Taper **Photograph 3, 65B, 50 ft in Advance of Taper**





Photograph 2, 65A Firth, 175 ft in Advance of Taper **Photograph 2, 65B, 175 ft in Advance of Taper**





Photograph 1, 65A Firth, 300 ft in Advance of Taper **Photograph 1, 65B, 300 ft in Advance of Taper**

Figure 11.3 Perspective Views of Type A and Type B Left-Turn Lanes at Two Study Sites

Further evidence that the taper should be at a relatively shallow angle is supported by some early data collection performed to estimate the path of left-turning vehicles on offset left-turn lanes for the purpose of locating speed detectors at appropriate locations on the tapered portion of the left-turn lane. The path of left-turn vehicle taken through gravel spread on left-turn lanes at unsignalized intersections for traction purposes was measured on Hwy 2 east of Lincoln, NE and Hwy 370 east of Omaha, NE in the winter of 2000. Figure 11.4 shows one of the left-turn lanes where data on gravel-cleared paths was collected.

Figure 11.4 Gravel-Cleared Paths on Offset Left-Turn Lanes

The posted speed on both expressways was 55 mph at the study locations. Taper rates of the cleared sand paths ranged from 8.4:1 to 15.6:1 on the left side of the left-turn lane and 15.6:1 to 23.6:1 on the right side of the left-turn lane. Table 11.2 shows the approximate taper rates of left-turning vehicles observed at these locations.

Table 11.2 Approximate Taper Rates of Left-Turners on Offset Left-Turn Lanes

Site Location	Length of Left-Turn Lane from Beginning of Taper to End of Storage Lane, ft	Paved Taper Rate of Left-Turn Lane, Longitudinal to Transverse, ft:ft	Left Cleared Gravel Path Taper Rate, Longitudinal to Transverse ft:ft	Right Cleared Gravel Path Taper Rate, Longitudinal to Transverse ft:ft
Hwy 2 and 84th St.	475	7:1	8.8:1	15.7:1
Hwy 2 and Pine Lake Rd.	450	7:1	8.4:1	23.6:1
Hwy 370 and 48th St.	475	8:1	11.3:1	15.6:1
Hwy 370 and 96th St.	575	8:1	15.6:1	23.1:1
Hwy 370 and 114th St.	600	8:1	13.2:1	23.4:1

Figure 11.5 shows the relationship between the longitudinal element of taper rate defined by the worn gravel path to the total length of the left-turn lane (taper and parallel storage lane) on the left and right sides of the left-turn lane.

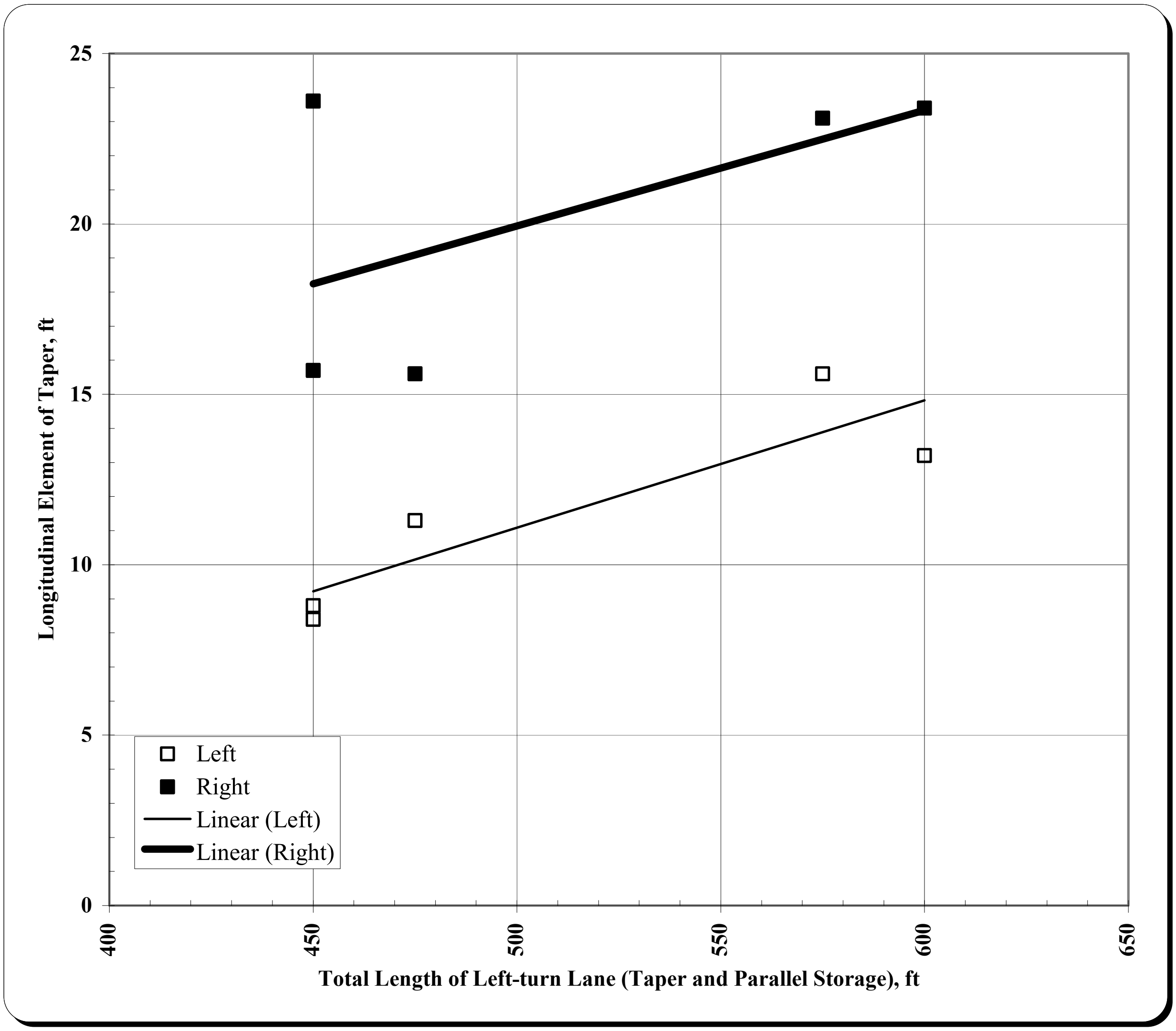


Figure 11.5 Relationship of Taper Rate to Total Left-Turn Lane Length Representing Cleared Gravel Path Tapers Worn by Left-Turn Vehicles

Equations 11.1 and 11.2 were developed from the relationship of data points in Figure 11.5.

$$TAPER_{LT} = 0.0373L - 7.5759 \qquad \textbf{(11.1)}$$

where,

$TAPER_{LT}$ = longitudinal element of taper rate on the left side of the cleared left-turn gravel path, ft, and

L = total length of left-turn lane (taper and storage), ft.

The adjusted R^2 value for Equation 11.1 is 0.79.

$$TAPER_{RT} = 0.0339L - 2.9892 \qquad \textbf{(11.2)}$$

where,

$TAPER_{RT}$ = longitudinal element of taper rate on the right side of the cleared gravel left-turn path, ft, and

L = total length of left-turn lane (taper and storage), ft.

The adjusted R^2 value for Equation 11.2 is 0.33.

The resulting left taper rate equation (when using the input value of 590 ft which represents the total length of the proposed Type AA left-turn lane with a minimum storage lane length of 70 ft) is 14.43:1. The resulting right taper rate is 22.99:1. The average value of these two rates is 18.71:1 which is near the proposed value of 20:1. The 20:1 taper is estimated to be a best fit for the worst-case situation of a 65-mph posted speed expressway.

5. **White painted left-turn arrow on pavement.** This arrow is located 160 ft beyond the beginning of the taper. At this location the left-turn lane is 8 ft wide and the 6 ft+ arrow is just able to fit within the lane with 1 ft of clearance to the lane edge on either side. This is further reinforcement that the left-turn lane is wide enough for a vehicle to enter (two-thirds of the full lane width). Desirably, this location is where braking should begin. Deceleration in advance of this point can be in the range of 1.1 to 3.9 fps^2 depending on the type of deceleration (in gear or reasonable braking). Although the 3.9 fps^2 value is in the high end of the reasonable braking range, this assumes that the driver doesn't begin braking until the beginning of the taper which would be a rare situation according to the data collected in this study. The expected speed of vehicles at this location is assumed to be 10 mph below the 95^{th}-percentile speed of 75 mph on a 65-mph posted speed rural expressway.
6. **Beginning of white through-lane painted separator.** The location of this point is at a 15-ft parallel offset from the tapered edge of the left-turn lane. This provides a visually wide lane width that should not intimidate the driver to adjust his/her deceleration rate. The left edge of the separator will taper to a 12-ft width at the beginning of the parallel storage lane and will provide a narrowing target for the entry to the parallel portion of the lane. White painted chevrons within the separator consist of a "V" shape, pointing at advancing traffic. The angle of the chevron is 45 degrees. The width of the separator is 14 ft when it is parallel to the storage lane. This is one foot less than the 15 ft "feasible allocation" given in NCHRP 375 (*9*) but the 14 ft dimension is the same as the current Type A design and is necessary to maximize the medial separator as explained in Item 9.
7. **End of the 20:1 taper and through-lane separator tapers, location of white painted left-turn arrow on pavement and word "ONLY".** The end of the painted taper is 520 ft beyond the beginning of the painted taper and is offset 26 ft from the edge of the passing lane. The speed at this point is assumed to be zero since the storage lane should be sized for a 5 percent exceedance according to the 1996 NDOR Roadway Design Manual (*1*). Braking is assumed to begin at a distance of 160 ft beyond the beginning of the taper so that leaves a distance of 360 ft for the deceleration to a full stop. The US customary braking distance equation in the 2001 Green Book (*4, p. 114*) is represented by Equation 11.3:

$$\text{Braking Distance} = \frac{V^2}{30\left(\left(\frac{a}{32.2}\right) \pm G\right)} \qquad \textbf{(11.3)}$$

where,

- V = initial speed, mph,
- A = constant deceleration rate of 11.2 fps^2, and
- G = percent profile grade of roadway divided by 100, (- for downgrade, + for upgrade).

Assuming a speed value which is 10 mph below the mean speed of all traffic in the passing lane of the expressway, the braking distance for 65-mph posted speed situation should be adequate for any situation. Table 11.3 shows the day/night mean speed at a distance of 900-1200 ft in advance of the taper. Speeds were averaged for the two posted speed groups and braking distances were calculated for both scenarios using speeds 10 mph less than the mean speed values (assumed to be the speed 160 beyond the beginning of the taper). The 20:1 taper length of 360 ft appears just adequate for the worst-case condition of a 65-mph posted speed on a 4 percent downgrade, which results in a braking distance of 352 ft.

Table 11.3 Braking Distances Calculated for Rural Expressway Conditions at Study Sites

Posted-Speed/Type	Day/Night Mean Speed, mph	Averaged Mean Speed, mph	10 mph Below Consolidated Mean Speed, mph	Grade, Percent	Required Braking Distance, ft 2001 Green Book (*4*)
55A	60.28	58 mph	48 mph	+4	198
55B	55.80	58 mph		-4	249
65A-Davey	66.61	67 mph			
65A-Firth	67.28	67 mph	57 mph	+4	280
65B	66.94	67 mph		-4	352

The left edge of the through-lane separator is 12 ft to the right of the end of the taper. According to research done by McCoy et al, vehicles tend to center themselves between painted left-turn lane lines, thus it is advantageous to keep this lane narrow to enhance the offset distance from the opposing left-turn lane. There is zero offset between the left edge of the storage lane and the right edge of the opposing lane, but if the driver's eye is assumed to be 5 ft from the left edge of the storage lane, the driver's eye is 5 ft offset from the right edge of the opposing left-turn lane. The minimum intersection sight distance for a worst-case situation (WB-62 combination truck on a 4 percent upgrade which is the steepest profile grade allowable on an expressway in Nebraska according to the 2002 Minimum Design Standards (*32*) turning left when opposed by a WB-62 combination truck) is 992 ft using the minimum gap acceptance formula from Case F of the 2001 Green Book intersection sight distance model (*4, p. 663*) shown in Equation 11.4.

$$ISD = 1.47\ V_{major}\ t_g \qquad \textbf{(11.4)}$$

where,

ISD = intersection sight distance (length of the leg of sight triangle along the major road), ft,
V_{major} = design speed of major road, mph, and
t_g = time gap for minor road vehicle to safely make a left-turn across all opposing lanes.

This 992 ft distance assumes:

- A width of 26 ft (2.17 equivalent lanes) of distance to cross at 0.7 seconds per lane in addition to the base time of 7.5 seconds (for crossing one lane) which totals 9.0 seconds, and
- If available, the 95th-percentile speed should be used to represent design speed field conditions as closely as possible. A 95th-percentile speed of 75 mph for a 65-mph posted speed expressway.

The Type AA design provides 1000 ft of sight distance along the major road which is adequate according to Case F of the gap acceptance model of the 2001 Green Book (*4, p. 678*).

If the design storage length is exceeded by left-turn arrivals, vehicles must store along the tapered section of the left-turn lane. The required intersection sight distance line of the left-turn driver's eye is not interrupted by vehicles stored at any point within the 20:1 left-turn lane taper.

NOTE: LEFT-TURN LANES ON CURVED HORIZONTAL ALIGNMENTS SHOULD BE CUSTOM DESIGNED TO EVALUATE SIGHT DISTANCE REQUIREMENTS AND SIGHT OBSTRUCTIONS PROPERLY.

8. **White painted left-turn arrow on pavement and word "ONLY".** The painted arrow is placed 1 ft in advance of the end of the storage lane length as a final reinforcement that the driver must turn left.
9. **End of asphalt surfacing (if expressway is constructed in concrete).** Delineation of the left-turn is no longer needed at this point. Actual turning movements within the length of the median opening will be made on the more resilient concrete pavement.
10. **Turf or textured pavement medial separator.** Desirably, the medial separator should be turf to reinforce the driver's perception that the Type AA left-turn lane is similar to a traditional left-turn lane like the one shown in Figure 8.7 for the Hwy 77 – Firth Road intersection. However, on steep grades, a turf separator may be difficult to establish and maintain. Gravel used for traction and salt used for melting ice in the winter months can accumulate and cause grass to be detrimentally affected. Medians without vegetation can erode and cause channels to develop in the soil, concentrating silt and water out onto the driving lanes (see Figure 11.3). A seed mixture resistant to salt should be used in the median from the beginning of the tapered segment to the paved part of the intersection. Special erosion control and drainage structures should also be included if required. If the

profile grade is greater than 2 percent, the medial separator should be surfaced with textured concrete. The contrast in color and texture will act to replicate features of the traditional left-turn lane. The end of the medial separator is tapered on both sides instead of rounded (similar to the gravel outline shown in Figure 11.6) to match wheel paths, for ease of construction and to ameliorate surfacing changes in the proximity.

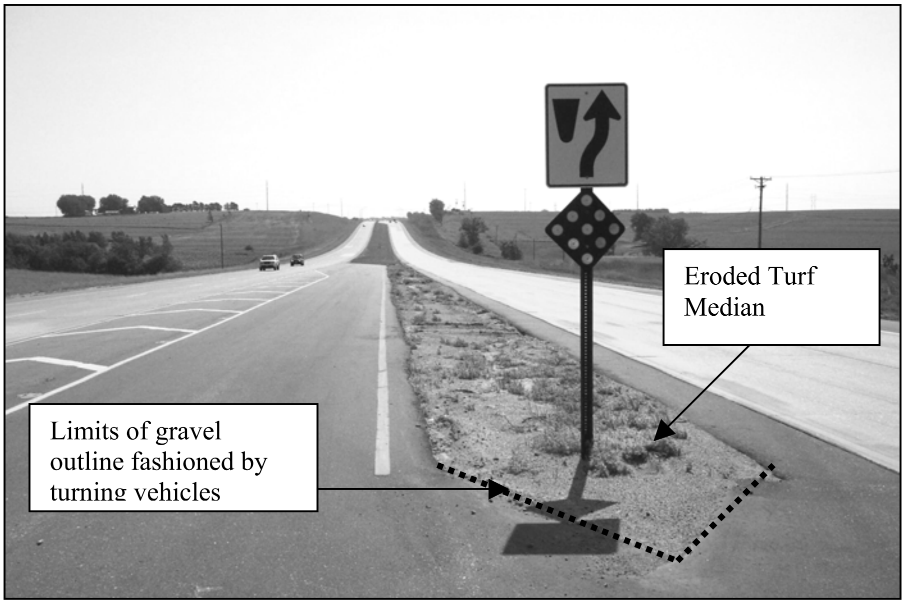


Figure 11.6 Turf Median Erosion on Steep Profile Grade

11. **Turning template of WB-62 fits 2 ft within pavement edge.** The Type AA design provides enough paved surface area to allow the wheels of a left-turning WB-62 combination truck to avoid the edge of surfacing by a distance of 2 ft both from the left-turn lane and from the cross road (shown as curved dashed lines at the intersection in Figure 11.1).

Recommendations for a Revised Traditional Left-Turn Lane, Type BB

Research findings concerning operations and safety at left-turn lanes on rural expressways also leads to the modification of geometric elements of Type B traditional left-turn lanes to better fit the expectations of drivers. The proposed design will henceforth be referred to as the Type BB left-turn design.

It is recommended that a 20:1 taper be used on the Type BB configuration for the same reasons stated previously for the Type AA configuration. Using the same braking distances calculated in Table 11.3, the Type BB design for a 65-mph posted speed expressway should include an additional 200 ft of parallel lane to accommodate braking from 57 mph (10 mph reduction from mean speed of 68 mph) at the location where the tapered lane width is 8 ft to 0 mph at the beginning point of the length made available for storage of left-turning vehicles. The Type BB design for a 55-mph posted speed expressway should provide an additional 100 ft of parallel storage length. The 20:1 taper from an 8 ft lane width to a 15 ft lane width provides 140 ft for deceleration within the taper. Figure 11.7 shows a typical Type BB configuration for both 55- and 65-mph designs.

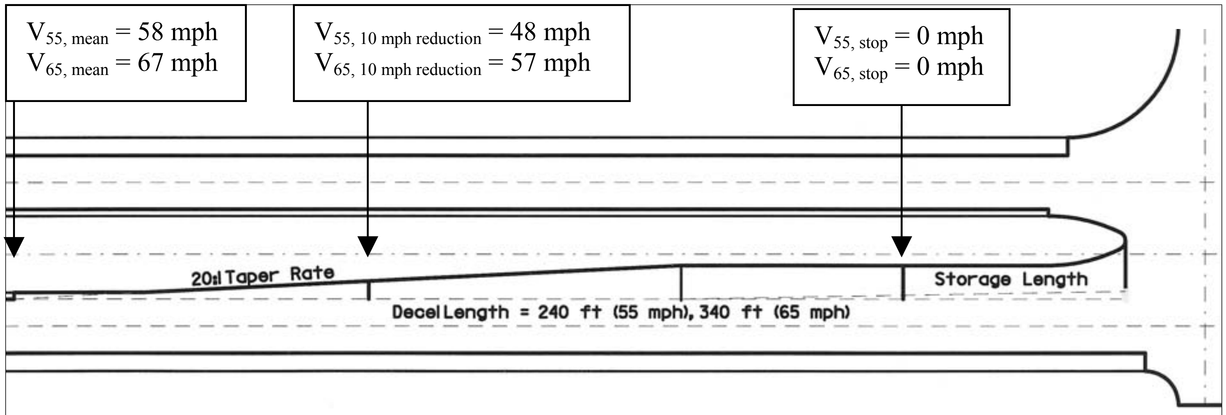


Figure 11.7 Recommendations for Geometrics of Type BB Configuration

The white paint line normally used to separate the left-turn lane from the through passing lane should be a line painted 15 ft from the left edge of the left-turn lane tapering to 12 ft from the left edge at the end of the island nose. This tapered paint line would provide a visually wide lane width that should not intimidate a left-turning driver into thinking he/she was entering a narrow lane but yet at the point where an opposing sight obstruction would become a problem, the driver would center his/her vehicle within the 12 ft lane, increasing the offset to the opposing left-turn lane. Similar striping has been used on arterials within the city of Lincoln, Nebraska based on previous research by McCoy, et al.

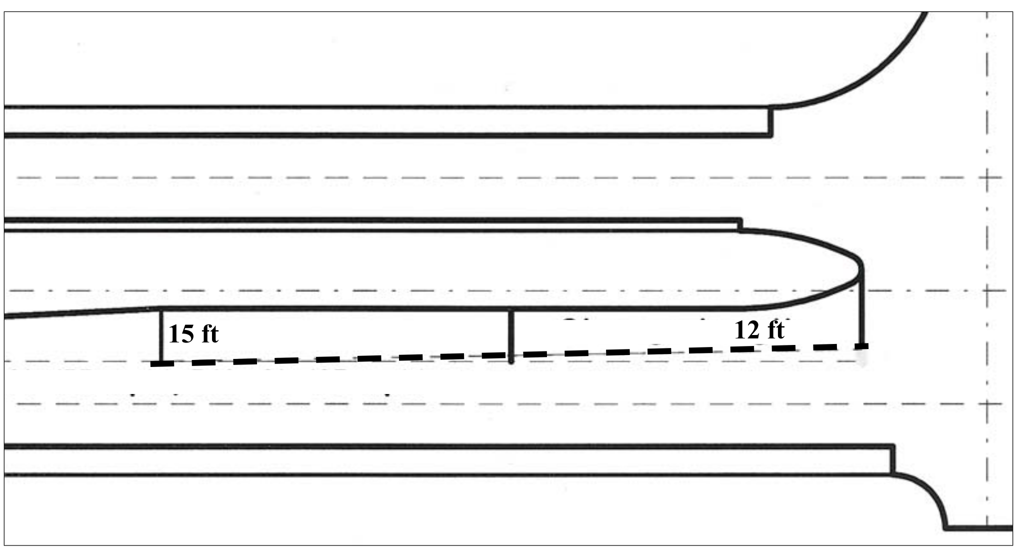


Figure 11.8 Tapered Left-Turn Lane Striping to Optimize Offset at Type BB Configurations

Special Considerations for Design of Left-Turn Lanes

The driver's perspective view should always be a factor in the design of any geometric roadway component. Care must be exercised to avoid locating the entrance to a left-turn lane just beyond an approaching crest. The driver must not be mislead into thinking the left-turn lane is an added lane. Left-turn lanes on horizontal curves must also be considered on an individual basis to avoid perceptual irregularities.

Need for Additional Research

Recommendations for enhancements to the current geometric criteria of the Type A and B left-turn lanes shown in the 1996 NDOR Roadway Design Manual are based on speculation that feature adjustments will improve safety at such locations. If recommendations are approved and Type AA and BB left-turn lanes are constructed, speed data should be collected in a similar manner to that of this research project to truly measure the success or failure of the revised geometrics.

Chapter 12
NDOR Implementation of Research Results

NDOR's implementation of the left-turn lane Research Project P537 is to change the taper rate of left-turn lanes on expressways from 15:1 to 20:1. The idea behind this is that the traffic should exit the left lane of the expressway at a higher speed, impeding the through traffic less and possibly lowering the number of rear-end crashes.

The NDOR Roadway Design Manual will be modified to show the new layout so that roadway designers will incorporate this into the expressway projects. The 20:1 will be built by the NDOR contractors and the traveling public will hopefully drive them as designed.

The NDOR Traffic Division will review the 20:1 taper for further signing if necessary and will be given the Excel spreadsheet to assist with recommendations to roadway designers. The NDOR Traffic Division will use this along with engineering judgment to determine the best solution for the intersection in question.

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Appendix A
Field Site Data

Appendix A

FIELD SITE DATA

The following is the detailed information of each field site. The information includes the traffic characteristics collected from the NUMETRIC NC-97 detectors, plan view of each intersection, with dimensions, detector positioning, and signing. The signing is dimensioned from the datum point.

Highway 77 and S-55E (Davey Road)

Posted Speed: 65

Intersection Type: A

Intersection Location: Highway 77 and S-55E (Davey Road)

Direction of Travel: NB

Nearest Location: Davey, NE

Dates of Study: May 9-10, 2001

Time of Study: 13:00 to 9:00

Daytime: 13:00 to 20:32, 6:14 to 9:00

Nighttime: 20:32 to 6:13

Weather: Clear

Pavement Condition: Dry

Number of Vehicles: 2761

Passenger Cars: 2469

Trucks: 292

% of all vehicles in passing lane: 21.55%

% of all vehicles in right lane: 78.45%

Number of Left-Turn Vehicles: 111

Passenger Cars: 97

Trucks: 14

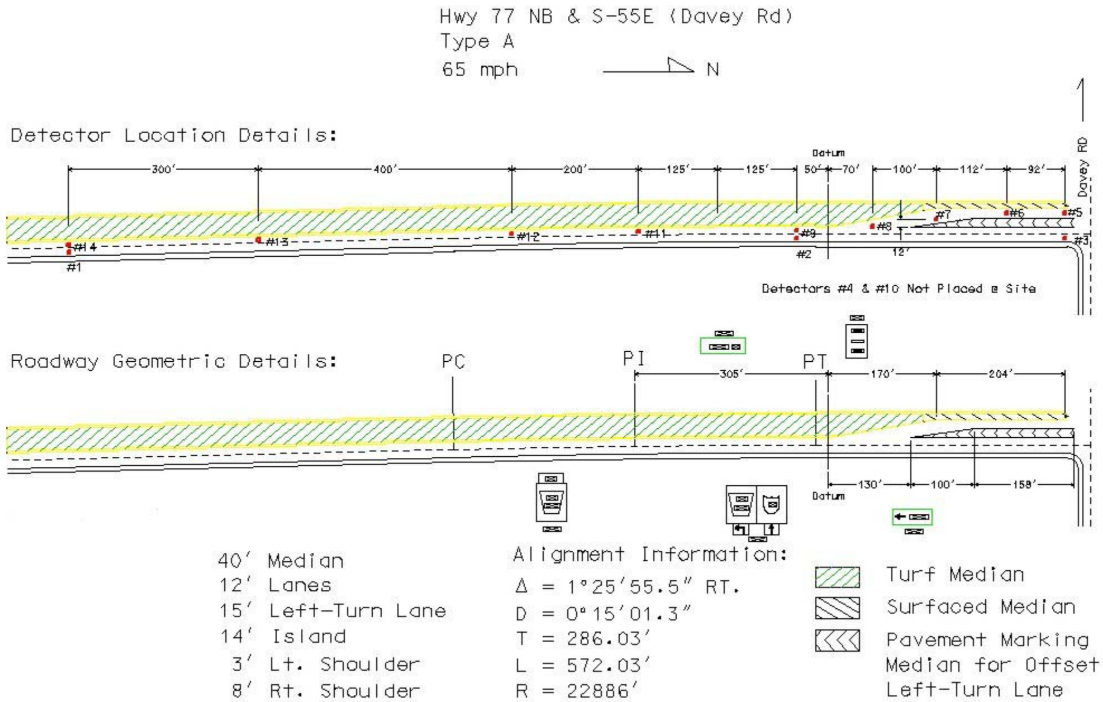


Figure B.1. Highway 77 & Davey Road Site Plan.

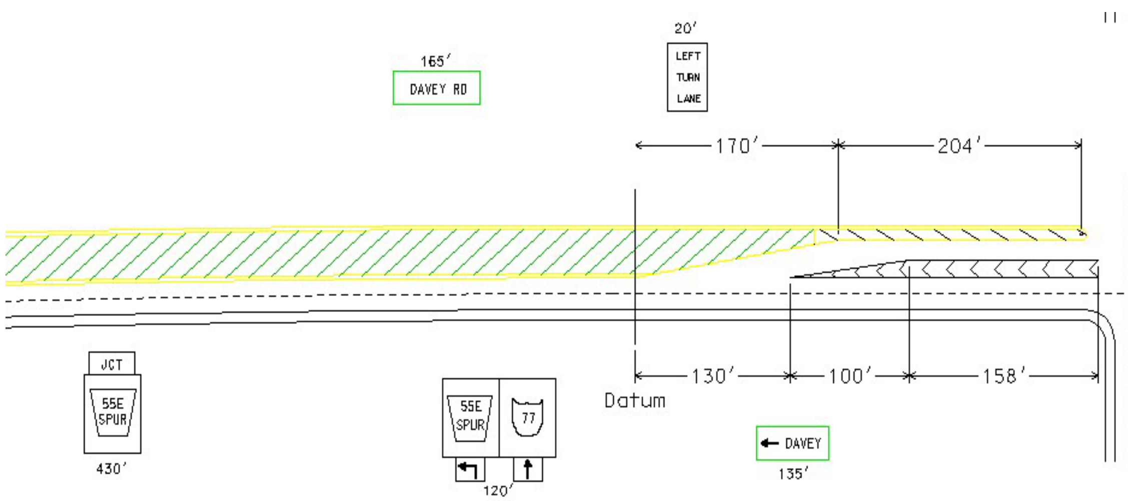


Figure B.2. Signing for Highway 77 & Davey Road Study Site.

Highway 77 and S-55H (Firth Road)

Posted Speed: 65

Intersection Type: A

Intersection Location: Hwy 77 and S-55H (Firth Road)

Direction of Travel: SB

Nearest Location: Firth, NE

Dates of Study: June 12-13, 2001

Time of Study: 13:00pm to 9:00am

Daytime: 13:00 to 20:57, 5:54 to 9:00

Nighttime: 20:58 to 5:53

Weather: Clear

Pavement Condition: Dry

Number of Vehicles: 3501

Passenger Cars: 3061

Trucks: 440

% of all vehicles in passing lane: 22.19%

% of all vehicles in right lane: 77.81%

Number of Left-Turn Vehicles: 126

Passenger Cars: 107

Trucks: 19

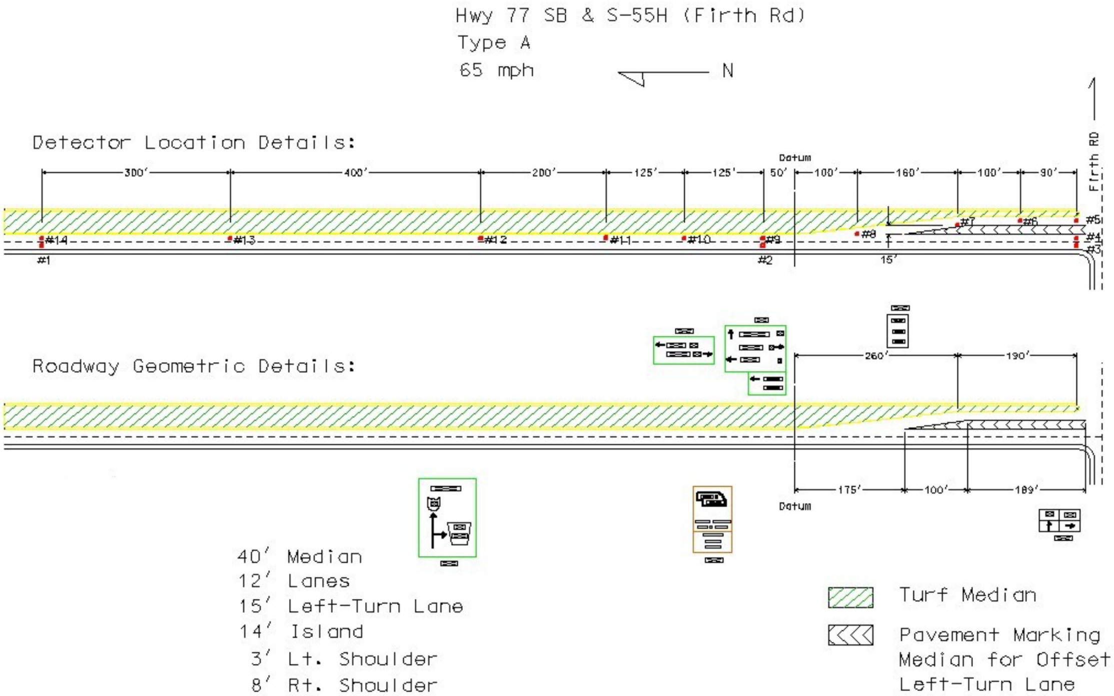


Figure B.3. Highway 77& Firth Road Site Plan.

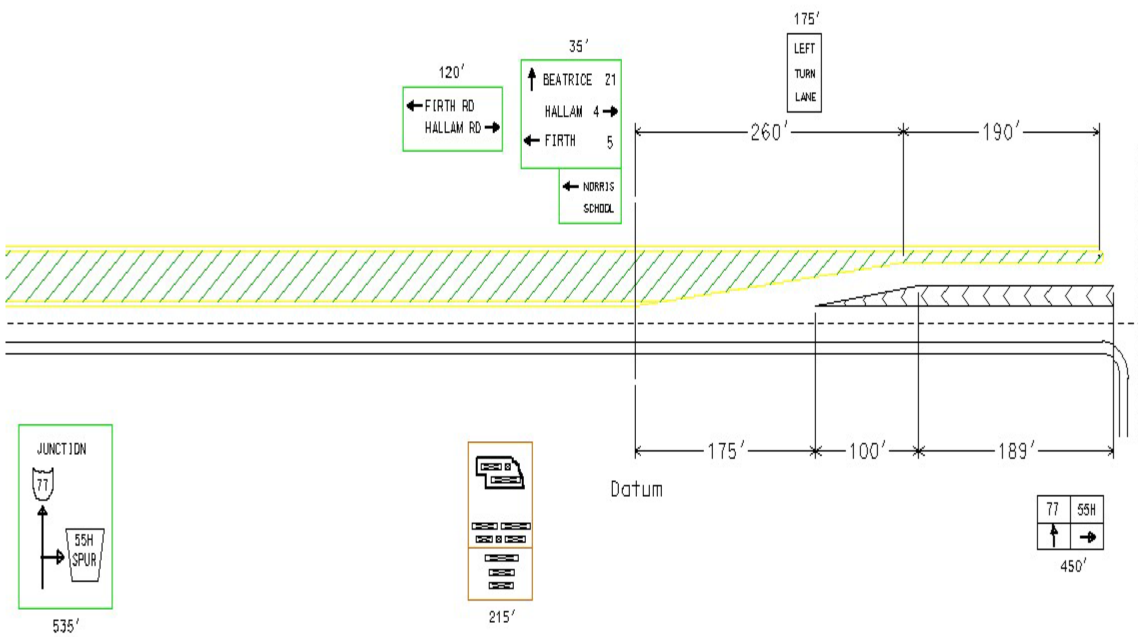


Figure B.4. Signing for Highway 77 & Firth Road Study Site.

Highway 2 and 84th St.

Posted Speed: 55

Intersection Type: A

Intersection Location: Highway 2 and 84th Street

Direction of Travel: EB

Nearest Location: Lincoln, NE

Dates of Study: June 18-19, 2001

Time of Study: 13:00 to 9:00

Daytime: 13:00 to 21:00, 5:55 to 9:00

Nighttime: 21:01 to 5:54

Weather: Clear (13:00 to 21:00), (23:00 to 9:00) Rain (21:00 to 23:00)

Pavement Condition: Dry (13:00 to 21:00), (23:00 to 9:00) Wet (21:00 to 23:00)

Number of Vehicles: 4644

Passenger Cars: 3854

Trucks: 790

% of all vehicles in passing lane: 37.34%

% of all vehicles in right lane: 66.26%

Number of Left-Turn Vehicles: 261

Passenger Cars: 237

Trucks: 24

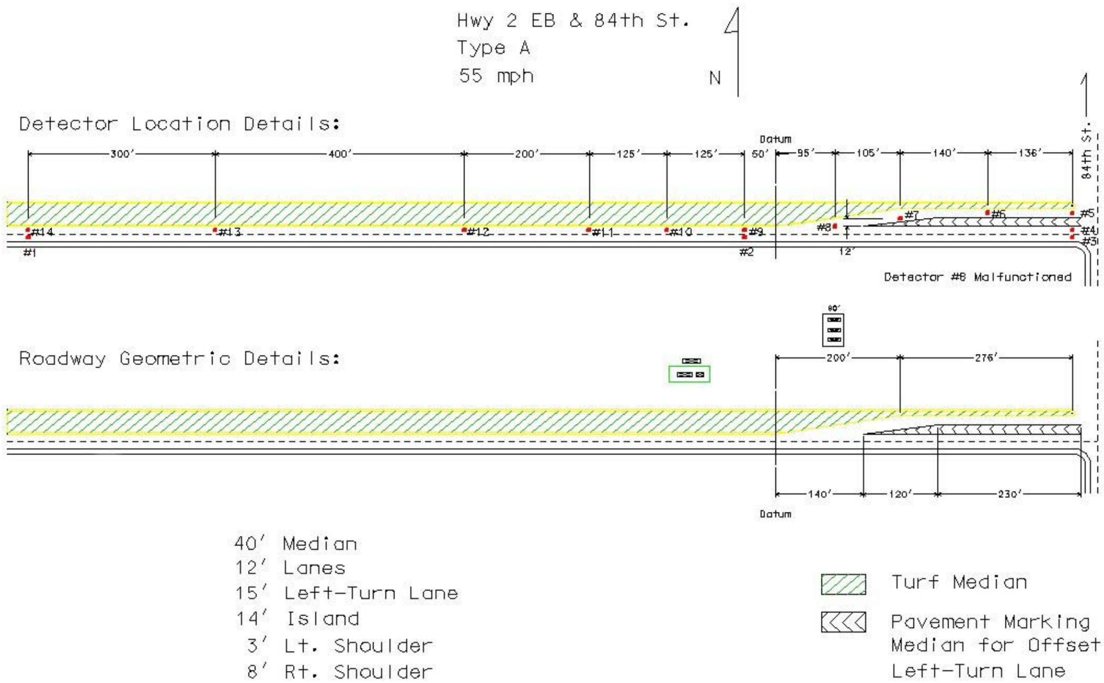


Figure B.5. Highway 2 & 84th Street Site Plan.

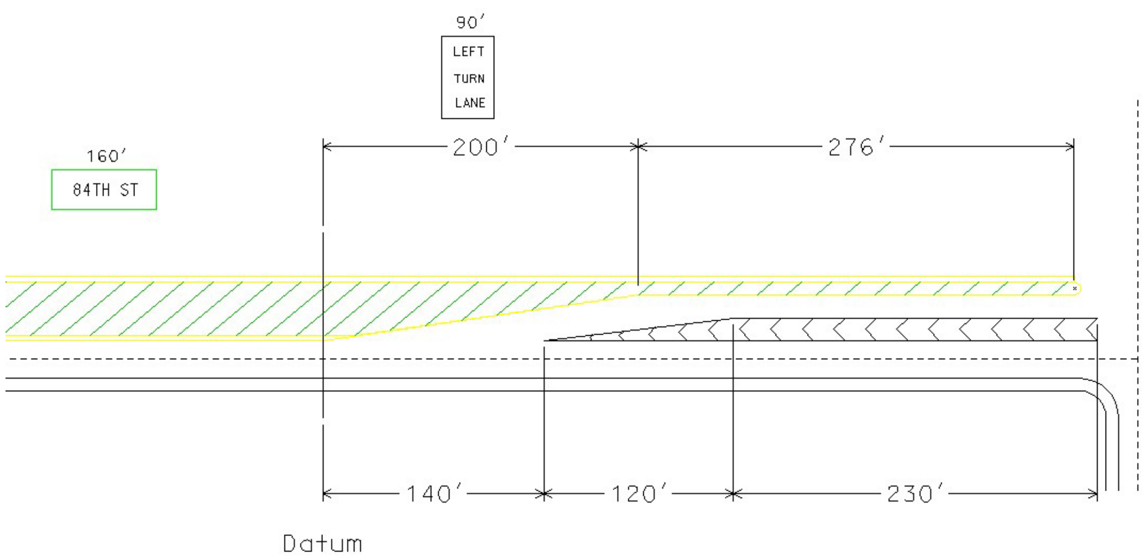


Figure B.6. Signing for Highway 2 & 84th Street Study Site.

Highway 2 and 148th St.

Posted Speed: 65

Intersection Type: B

Intersection Location: Hwy 2 and 148th St.

Direction of Travel: EB

Nearest Location: Bennet, NE

Dates of Study: June 21-22, 2001

Time of Study: 13:00pm to 9:00am

Daytime: 13:00 to 21:02, 5:55 to 9:00

Nighttime: 21:03 to 5:54

Weather: Clear

Pavement Condition: Dry

Number of Vehicles: 4739

Passenger Cars: 3920

Trucks: 819

% of all vehicles in passing lane: 21.04%

% of all vehicles in right lane: 78.96%

Number of Left-Turn Vehicles: 115

Passenger Cars: 101

Trucks: 14

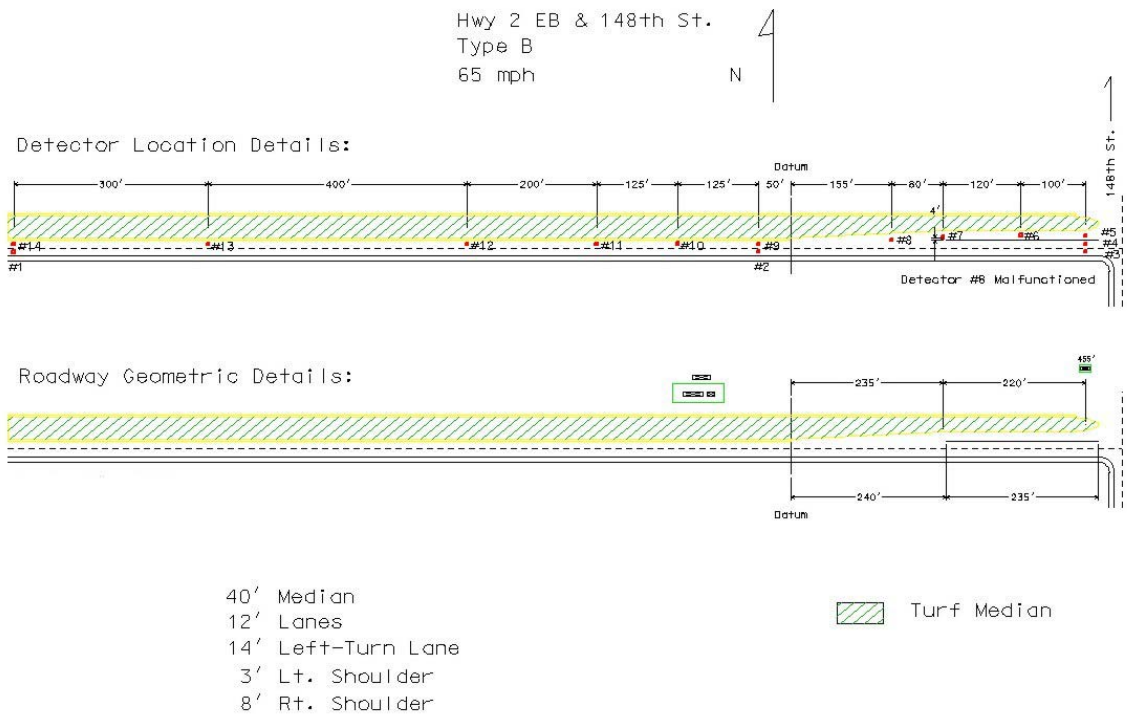


Figure B.7. Highway 2 & 148th Street Site Plan.

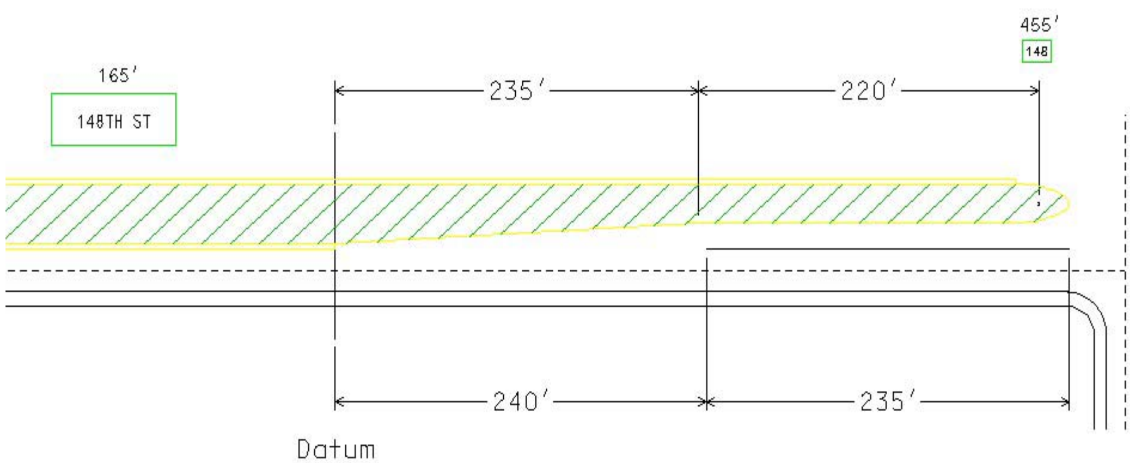


Figure B.8. Signing for Highway 2 & 148th Street Study Site.

Highway 81 and S. 21st St.

Posted Speed: 55

Intersection Type: B

Intersection Location: Highway 81 and South 21st Street

Direction of Travel: SB

Nearest Location: York, NE

Dates of Study: July 10-11, 2001

Time of Study: 13:00 to 9:00

Daytime: 13:00 to 21:02, 6:08 to 9:00

Nighttime: 21:03 to 6:07

Weather: Clear

Pavement Condition: Dry

Number of Vehicles: 4471

Passenger Cars: 4016

Trucks: 455

% of all vehicles in passing lane: 42.00%

% of all vehicles in right lane: 58.00%

Number of Left-Turn Vehicles: 179

Passenger Cars: 165

Trucks: 14

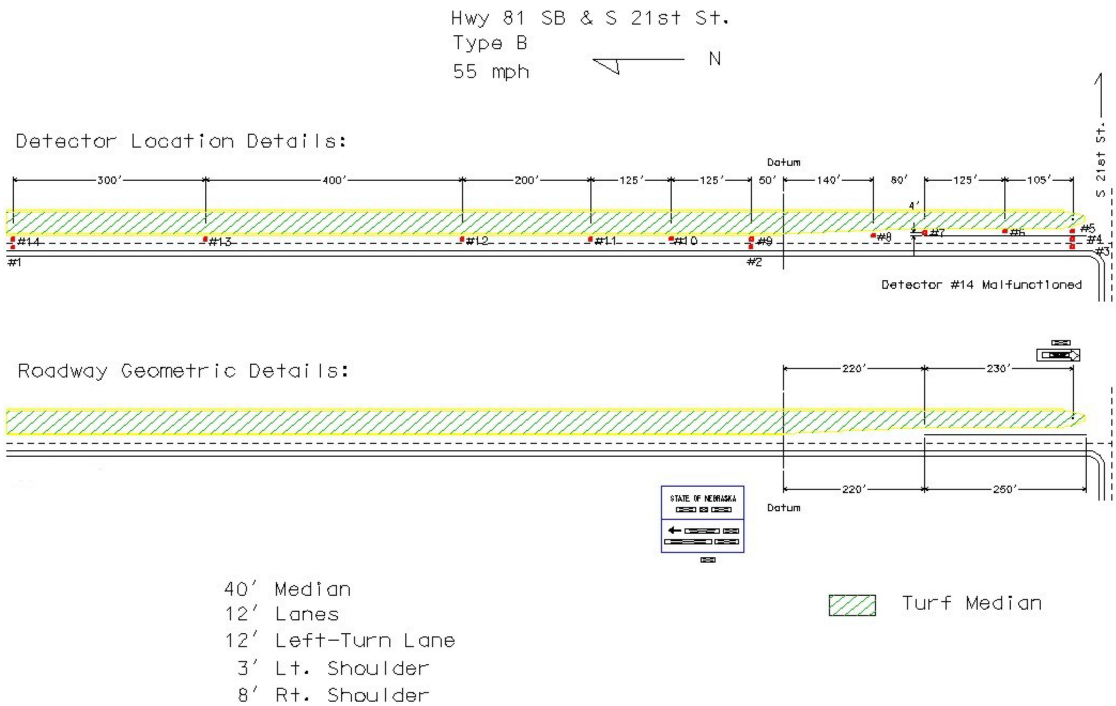


Figure B.9. Highway 81 & S. 21st Street Site Plan.

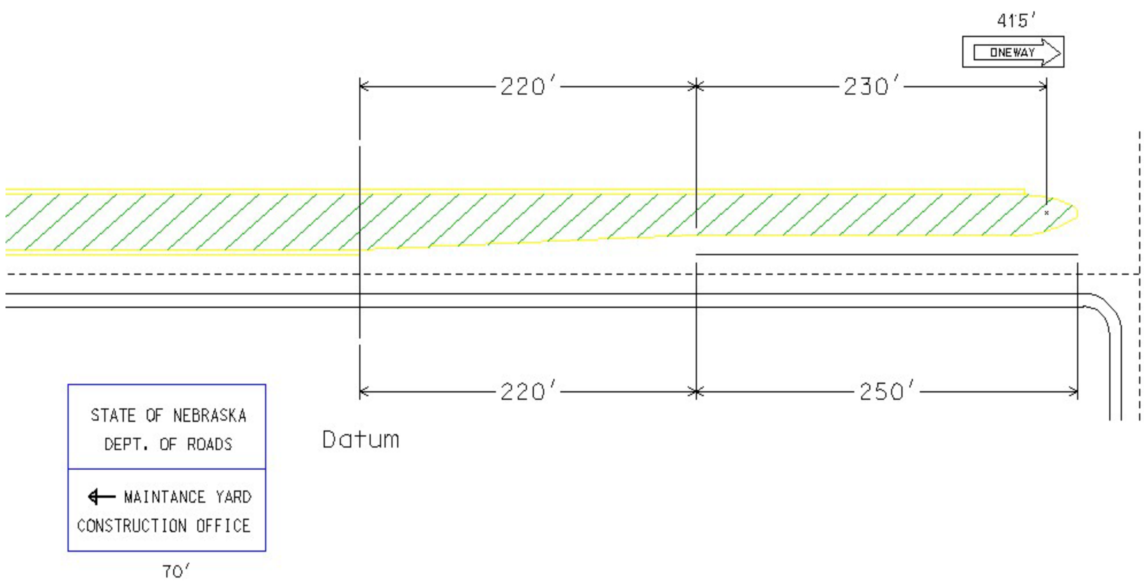


Figure B.10. Signing for Highway 81 & S. 21st Street Study Site.

Appendix B
Instructions for Use of Type A/B Left-Turn Lane Decision Spreadsheets

Appendix B

Instructions for Use of Type A/B Left-Turn Lane Decision Spreadsheet

TYPE A COST EVALUATION

Refer to Chapters 9 and 10 of the research report for a more detailed explanation of the following calculations.

NOTE: Spreadsheet calculations are based on the following annualization factor values:
Traffic growth rate: 2.5% per year; Project design life: 20 years; Discount rate: 4%;
Salvage value: $0.

Worksheet Title:

Type A Total Costs

Description of Worksheet:

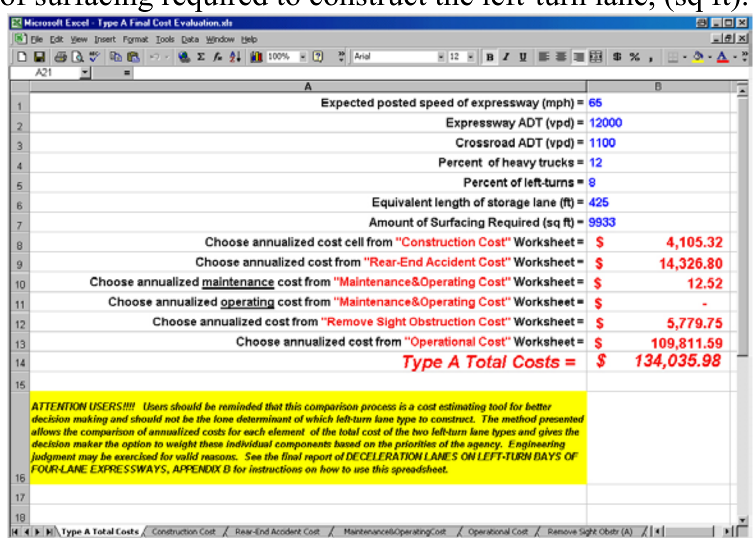
This sheet is the location where input values are entered to determine the operational, safety (rear-end and left-turn-leaving accidents), construction, and maintenance/operating costs of Type A left-turn lanes.

The resulting costs for each item are drawn from calculations made on following worksheets and the cell labeled "Type A Total Costs " sums all cost categories to arrive at a Type A left-turn cost.

NOTE: Some cost values must be added to this worksheet manually to arrive at the appropriate final costs. Instructions to alert the designer to this manual insertion are included in the "Designer Generated Spreadsheet Cell Change" section of each applicable worksheet.

Roadway Designer Generated Input Values:

- Expected posted speed of expressway (mph),
- Current expressway AADT at construction completion year, both directions, (vpd),
- Current crossroad AADT at construction completion year, both directions, (vpd),
- Percent heavy trucks in total approach volume, (%),
- Percent of left-turns in total approach volume, (%),
- Equivalent length of storage lane, (lineal ft),
- Amount of surfacing required to construct the left-turn lane, (sq ft).

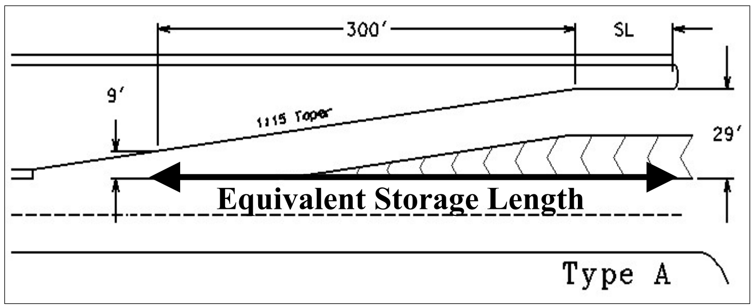


	A	B
1	Expected posted speed of expressway (mph) =	65
2	Expressway ADT (vpd) =	12000
3	Crossroad ADT (vpd) =	1100
4	Percent of heavy trucks =	12
5	Percent of left-turns =	8
6	Equivalent length of storage lane (ft) =	425
7	Amount of Surfacing Required (sq ft) =	9933
8	Choose annualized cost cell from "Construction Cost" Worksheet =	$ 4,105.32
9	Choose annualized cost from "Rear-End Accident Cost" Worksheet =	$ 14,326.80
10	Choose annualized maintenance cost from "Maintenance&Operating Cost" Worksheet =	$ 12.52
11	Choose annualized operating cost from "Maintenance&Operating Cost" Worksheet =	$ -
12	Choose annualized cost from "Remove Sight Obstruction Cost" Worksheet =	$ 5,779.75
13	Choose annualized cost from "Operational Cost" Worksheet =	$ 109,811.59
14	*Type A Total Costs =*	*$ 134,035.98*
15		
16	*ATTENTION USERS!!!! Users should be reminded that this comparison process is a cost estimating tool for better decision making and should not be the lone determinant of which left-turn lane type to construct. The method presented allows the comparison of annualized costs for each element of the total cost of the two left-turn lane types and gives the decision maker the option to weight these individual components based on the priorities of the agency. Engineering judgment may be exercised for valid reasons. See the final report of DECELERATION LANES ON LEFT-TURN BAYS OF FOUR-LANE EXPRESSWAYS, APPENDIX B for instructions on how to use this spreadsheet.*	
17		
18		

Definitions:

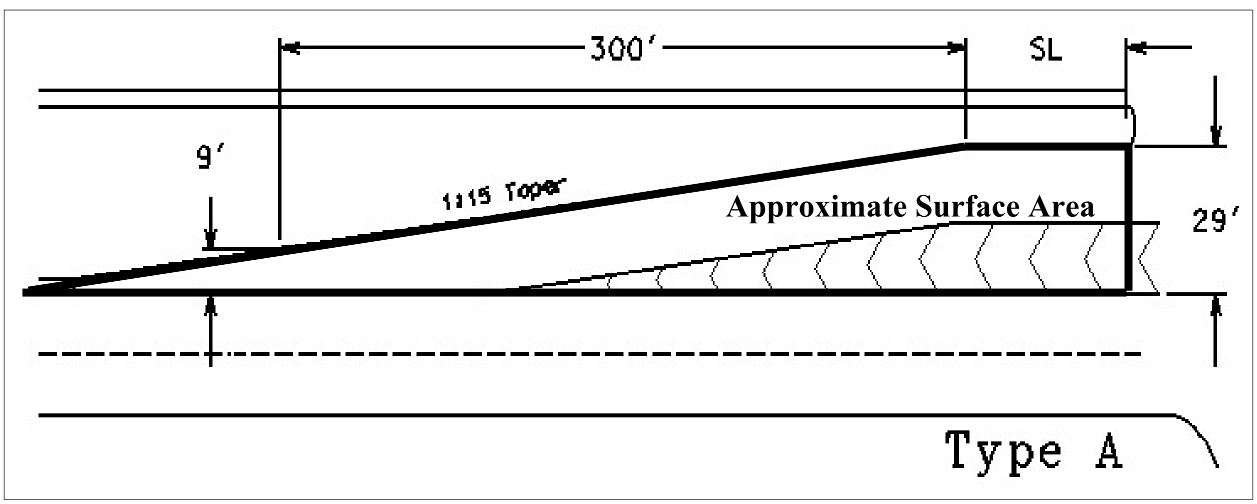
Equivalent length of storage lane:

The equivalent length of storage lane is defined in the research project as the longitudinal length in lineal feet of a left-turn lane which can store vehicles safely beyond the edge of the adjacent through traffic lane. The distance is calculated by determining the longitudinal distance along the tapered segment greater than 9 ft wide plus the length of parallel lane required for additional deceleration length and/or storage of left-turning vehicles to the end of the median island nose radius point.



Amount of surfacing required to construct left-turn lane:

This is an approximate estimate of the surface area of the left-turn lane beyond the edge of the through traffic lane in units of square feet. The calculations should represent the tapered portion of the left-turn lane and the parallel portion of the lane up to the end of the median island nose radius point.



Worksheet Title:

Construction Cost

Description of Worksheet:

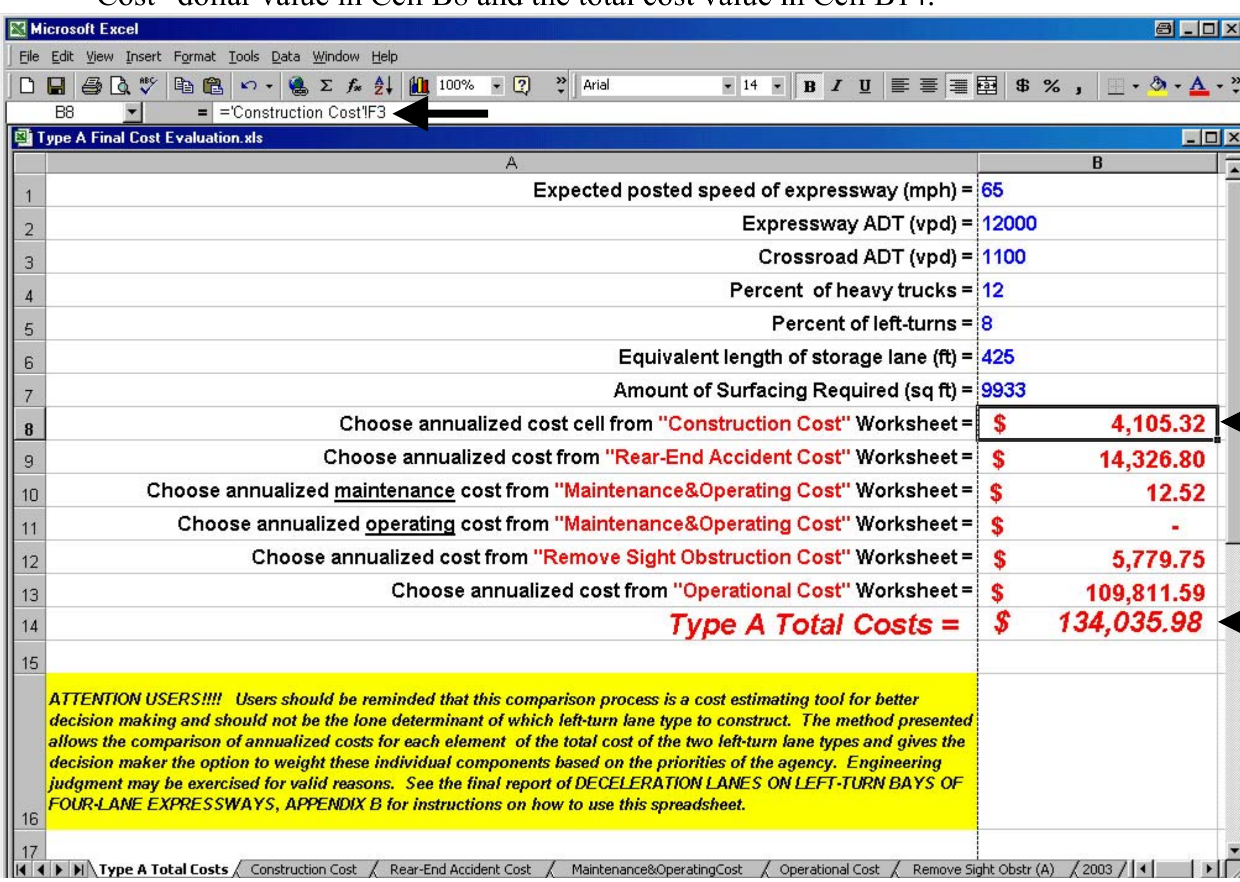
This sheet provides the annualized cost of construction of a Type A left-turn lane based on the equivalent storage length given on the worksheet titled "Type A Total Costs" and Equations 9.16 to 9.23 established for construction costs in the research report.

Roadway Designer Generated Input Values:

- Type of project, (large or small retrofit),
- Type of material to be used for construction of the left-turn lane, (concrete or asphalt), and
- Lighting or no lighting.

Roadway Designer Generated Spreadsheet Cell Change:

The designer must determine which annualized cost applies to his/her input values stated above and note the cell column and row which includes the value. The designer must then return to the "Type A Total Costs" worksheet highlighting the Construction Cost Cell B8 and in the "Edit Formula" line, replace the current cell information with the cell information that applied to his/her input values. This will change the "Construction Cost" dollar value in Cell B8 and the total cost value in Cell B14.



Worksheet Title:

Rear-End Accident Cost

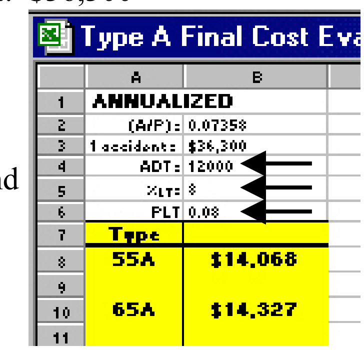
Description of Worksheet:

This sheet provides the annualized cost of potential rear-end accidents that may occur due to speed differentials between left-turning and through vehicles in the passing lane of the expressway. Values are calculated using the following assumptions:

- Portion of total approach traffic in the passing lane: 55 mph posted speed = 0.38,
- Portion of total approach traffic in the passing lane: 65 mph posted speed = 0.22,
- Left-turn deceleration distance in through traffic passing lane: 450 ft, and
- Societal cost for rear-end accident on an expressway in Nebraska: $36,300

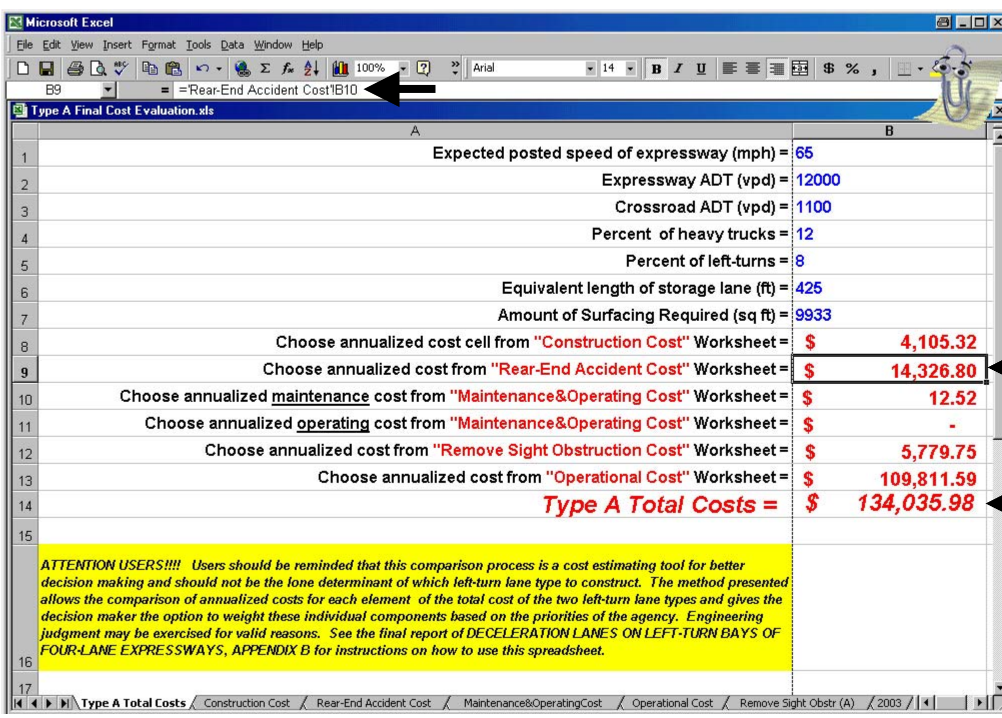
Roadway Designer Generated Input Values:

- Current expressway AADT at construction completion year, both directions in Cell B4, (vpd),
- Percent of left-turns in total approach volume in Cell B5, (%), and
- Portion of left-turns in total approach volume in Cell B6, (% divided by 100).



Roadway Designer Generated Spreadsheet Cell Change:

The designer must determine which annualized cost applies to his/her posted speed category and note the cell column and row which includes the value (Cell B8 or B10). The designer must then return to the "Type A Total Cost" worksheet highlighting the Rear-End Accident Cost Cell B9 and in the "Edit Formula" line, replace the current cell information with the cell information that applies to his/her input values. This will change the "Rear-End Accident Cost" dollar values in Cell B9 and the total cost value in Cell B14.



Worksheet Title:

Maintenance & Operating Cost

Description of Worksheet:

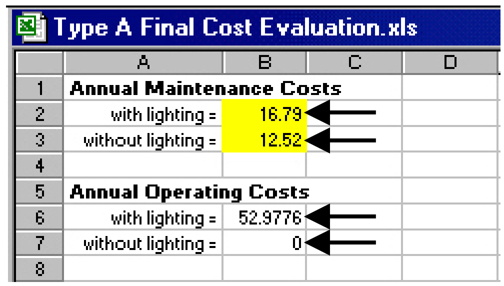
This sheet provides the annualized cost of surfacing maintenance and maintenance/operating costs of lighting if applicable. Values are calculated using the following assumptions:

- Annual surfacing maintenance costs of expressway pavement: $0.00126 per sq ft,
- Annual lighting maintenance: $400 per installation, and
- Annual lighting power costs: $720.

Roadway Designer Generated Input Values:

- Lighting or no lighting.

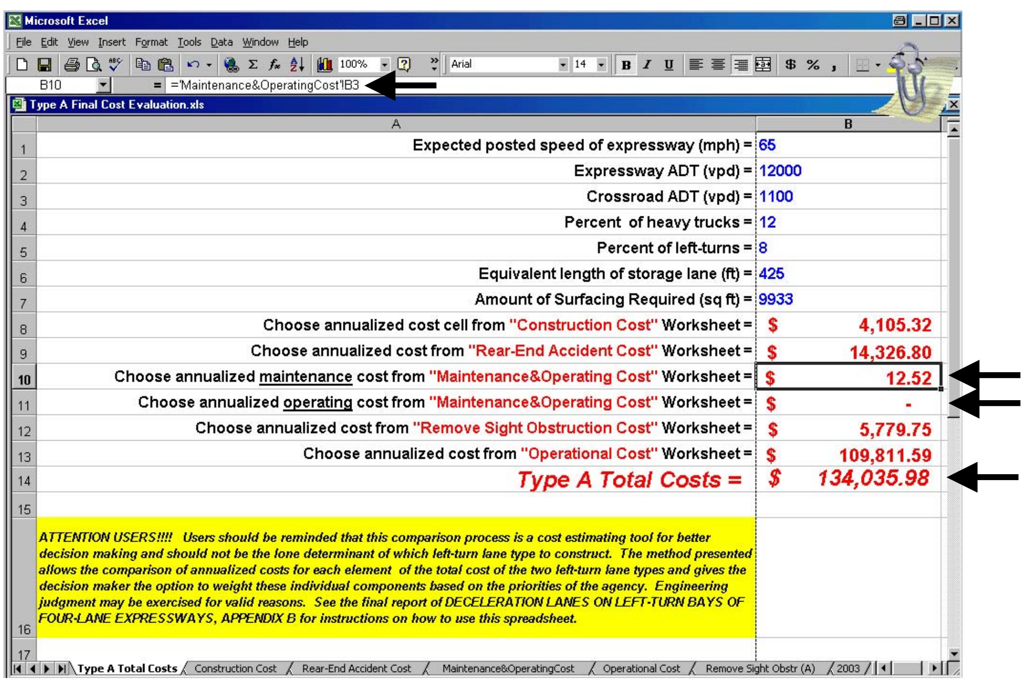
Roadway Designer Generated Spreadsheet Cell Change:



Type A Final Cost Evaluation.xls

	A	B	C	D
1	Annual Maintenance Costs			
2	with lighting =	16.79		
3	without lighting =	12.52		
4				
5	Annual Operating Costs			
6	with lighting =	52.9776		
7	without lighting =	0		
8				

The designer must determine which annualized cost applies to his/her lighting category and note the cell column and row which includes the value. Note that the maintenance costs are separated from the operating costs and are shown on two lines of the "Type A Total Costs" worksheet. The designer must then return to the "Type A Total Cost" worksheet highlighting both the maintenance and operating cells separately and in the appropriate "Edit Formula" line, replace the current cell information with the cell information that applies to his/her input values. Cell B2 calculates maintenance costs of surfacing and lighting and Cell B6 calculates operating costs of lighting. Cell B3 calculates maintenance costs of surfacing only and Cell B7 is zero since there is no lighting to operate.



Microsoft Excel

B10 = ='Maintenance&OperatingCost'!B3

Type A Final Cost Evaluation.xls

	A	B
1	Expected posted speed of expressway (mph) =	65
2	Expressway ADT (vpd) =	12000
3	Crossroad ADT (vpd) =	1100
4	Percent of heavy trucks =	12
5	Percent of left-turns =	8
6	Equivalent length of storage lane (ft) =	425
7	Amount of Surfacing Required (sq ft) =	9933
8	Choose annualized cost cell from "Construction Cost" Worksheet =	$ 4,105.32
9	Choose annualized cost from "Rear-End Accident Cost" Worksheet =	$ 14,326.80
10	Choose annualized maintenance cost from "Maintenance&Operating Cost" Worksheet =	$ 12.52
11	Choose annualized operating cost from "Maintenance&Operating Cost" Worksheet =	$ -
12	Choose annualized cost from "Remove Sight Obstruction Cost" Worksheet =	$ 5,779.75
13	Choose annualized cost from "Operational Cost" Worksheet =	$ 109,811.59
14	*Type A Total Costs =*	*$ 134,035.98*
15		
16	*ATTENTION USERS!!!! Users should be reminded that this comparison process is a cost estimating tool for better decision making and should not be the lone determinant of which left-turn lane type to construct. The method presented allows the comparison of annualized costs for each element of the total cost of the two left-turn lane types and gives the decision maker the option to weight these individual components based on the priorities of the agency. Engineering judgment may be exercised for valid reasons. See the final report of DECELERATION LANES ON LEFT-TURN BAYS OF FOUR-LANE EXPRESSWAYS, APPENDIX B for instructions on how to use this spreadsheet.*	
17		

Type A Total Costs / Construction Cost / Rear-End Accident Cost / Maintenance&OperatingCost / Operational Cost / Remove Sight Obstr (A) / 2003

Worksheet Title:

Operational Cost

Description of Worksheet:

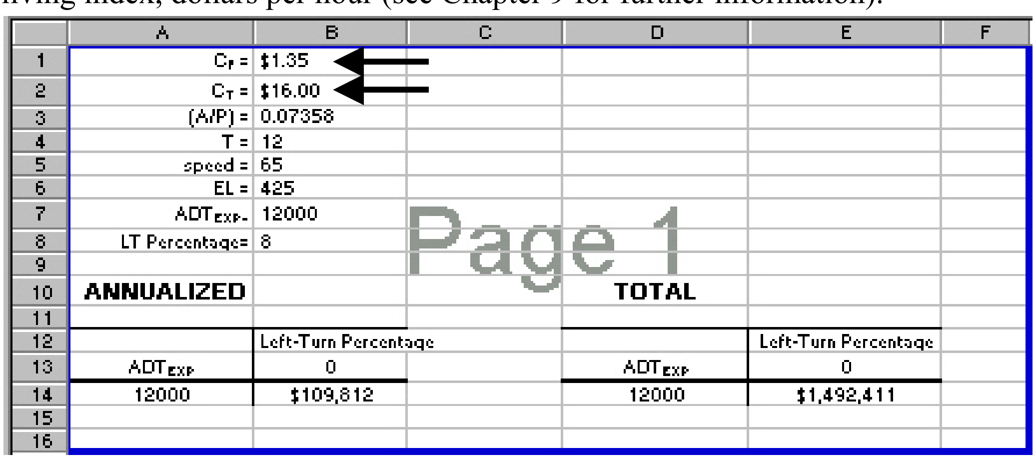
This sheet provides the annualized cost of delay and fuel consumption for the vehicles using the approach during the design life of the project. Values are calculated using the following assumptions:

- Average hourly portions of daily traffic shown in Table 8.6 of research report,
- Equations 7.1, 7.2, and 9.1-9.3 from the research report,
- Cost of 1 gallon of gasoline: $1.35, and
- Unit cost value of travel time: $16.00 per hour,

Note: Twenty separate spreadsheet pages were required to do this calculation similar to the worksheet titled "2003" for each of the 20 years of the design life of the project, using the percent of total AADT in each of the 24 hours of the day to calculate hourly operational costs. These worksheets are hidden since they occupy so much room. Calculations from the worksheets are used to arrive at the final operational costs. To unhide any of these worksheets, click on Format>Sheet>Unhide. Hidden sheets are listed by year and may be unhidden by double-clicking the name of the hidden worksheet you want to display. The dates of the worksheets are representing the number of years in the analysis which is assumed to be 20 so the 2003 tab represents costs from Year 1 of the analysis.

Roadway Designer Generated Input Values:

- Current price of gasoline, dollars per gallon, and
- Unit cost value of travel time, updated from 1975 to the current year using the cost of living index, dollars per hour (see Chapter 9 for further information).



	A	B	C	D	E	F
1	C_F =	$1.35				
2	C_T =	$16.00				
3	(A/P) =	0.07358				
4	T =	12				
5	speed =	65				
6	EL =	425				
7	ADT_{EXP} =	12000				
8	LT Percentage=	8				
9						
10	ANNUALIZED			TOTAL		
11						
12		Left-Turn Percentage			Left-Turn Percentage	
13	ADT_{EXP}	0		ADT_{EXP}	0	
14	12000	$109,812		12000	$1,492,411	
15						
16						

Roadway Designer Generated Spreadsheet Cell Change:

Unlike previous worksheets, the annualized value for operational costs will always result in cell B13 and will automatically be changed on the main "Type A Total Cost" worksheet. The sum of annualized costs will automatically be updated in cell B14.

Worksheet Title:

Remove Sight Obstr A

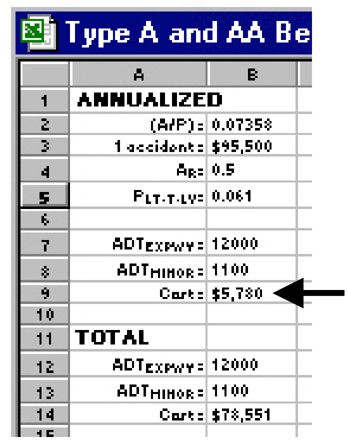
Description of Worksheet:

This sheet provides the annualized cost of left-turn-leaving accidents that are expected to occur with Type A offset left-turn lanes. Values are calculated using the following assumptions:

- Societal cost for rear-end accident on an expressway in Nebraska: $95,500,
- Accident reduction factor for offset left-turn lanes: 0.50,
- Portion left-turn-leaving accidents are of total expressway accidents in Nebraska: 0.061.

Roadway Designer Generated Input Values:

No values need to be changed on this worksheet since all volume data is automatically selected from the "Type A Total Cost" worksheet.

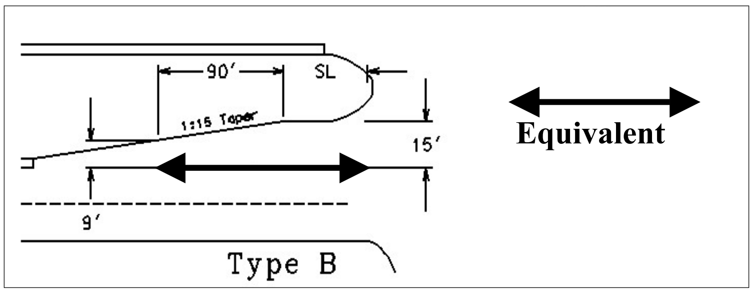


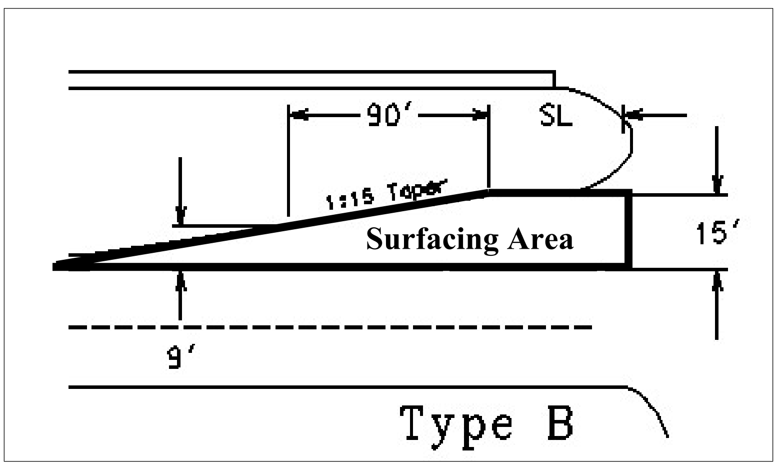
Roadway Designer Generated Spreadsheet Cell Change:

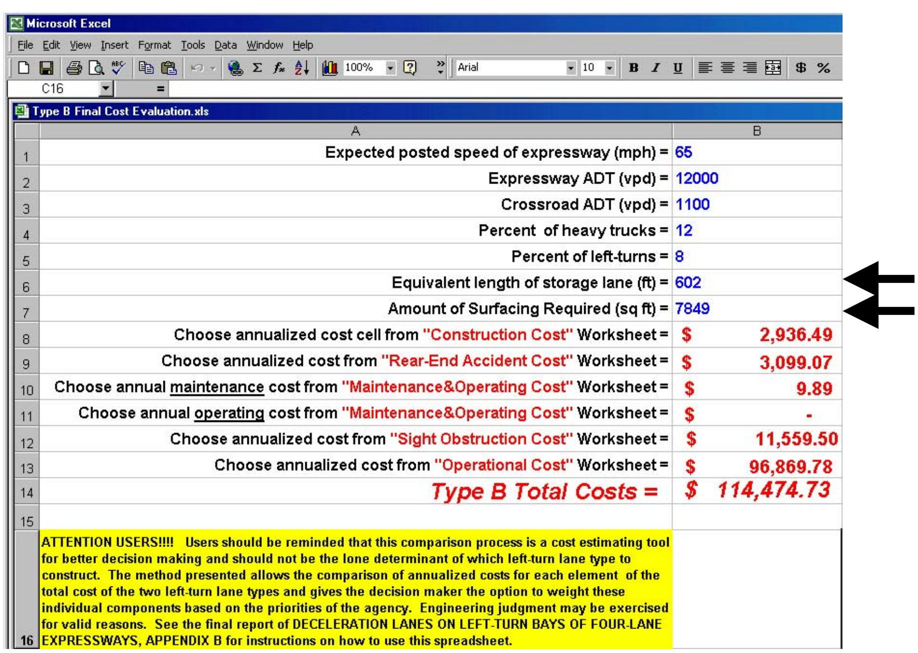
The annualized value for operational costs will always result in cell B9 and will automatically be changed on the main "Type A Total Cost" worksheet in cell B12. The sum of annualized costs will automatically be updated in cell B14.

Instructions for Use of Type A/B Left-Turn Lane Decision Spreadsheet
TYPE B COST EVALUATION

The process of using the spreadsheet to estimate the annualized costs of a Type B left-turn lane is the same as the Type A left-turn lane with the exception that the equivalent storage length and surfacing area entered into the "Type B Costs" worksheet must reflect the Type B design. Type B costs reflect safety and operational characteristics determined in the research.






Microsoft Excel

Type B Final Cost Evaluation.xls

	A	B
1	Expected posted speed of expressway (mph) =	65
2	Expressway ADT (vpd) =	12000
3	Crossroad ADT (vpd) =	1100
4	Percent of heavy trucks =	12
5	Percent of left-turns =	8
6	Equivalent length of storage lane (ft) =	602
7	Amount of Surfacing Required (sq ft) =	7849
8	Choose annualized cost cell from "Construction Cost" Worksheet =	$ 2,936.49
9	Choose annualized cost from "Rear-End Accident Cost" Worksheet =	$ 3,099.07
10	Choose annual maintenance cost from "Maintenance&Operating Cost" Worksheet =	$ 9.89
11	Choose annual operating cost from "Maintenance&Operating Cost" Worksheet =	$ -
12	Choose annualized cost from "Sight Obstruction Cost" Worksheet =	$ 11,559.50
13	Choose annualized cost from "Operational Cost" Worksheet =	$ 96,869.78
14	*Type B Total Costs =*	*$ 114,474.73*
15		
16	ATTENTION USERS!!!! Users should be reminded that this comparison process is a cost estimating tool for better decision making and should not be the lone determinant of which left-turn lane type to construct. The method presented allows the comparison of annualized costs for each element of the total cost of the two left-turn lane types and gives the decision maker the option to weight these individual components based on the priorities of the agency. Engineering judgment may be exercised for valid reasons. See the final report of DECELERATION LANES ON LEFT-TURN BAYS OF FOUR-LANE EXPRESSWAYS, APPENDIX B for instructions on how to use this spreadsheet.	