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ACCESS CONTROL DESIGN ON HIGHWAY INTERCHANGES

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Abstract

The adequate spacing and design of access to crossroads in the vicinity of freeway ramps are critical to the safety and traffic operations of both the freeway and the crossroad. The research presented in this report develops a methodology to evaluate the safety impact of different access road spacing standards. The results clearly demonstrate the shortcomings of the AASHTO standards and the benefits of enhancing them. The models developed as part of this research were used to compute the crash rate associated with alternative section spacing. The study demonstrates that the models satisfied the statistical requirements and provide reasonable crash estimates. The results demonstrate an eight-fold decrease in the crash rate when the access road spacing increases from 0 to 300 m. An increase in the minimum spacing from 90 m (300 ft) to 180 m (600 ft) results in a 50 percent reduction in the crash rate. The models were used to develop lookup tables that quantify the impact of access road spacing on the expected number of crashes per unit distance. The tables demonstrate a decrease in the crash rate as the access road spacing increases.

An attempt was made to quantify the safety cost of alternative access road spacing using a weighted average crash cost. The weighted average crash cost was computed considering that 0.6, 34.8, and 64.6 percent of the crashes were fatal, injury, and property damage crashes, respectively. These proportions were generated from the field observed data. The cost of each of these crashes was provided by VDOT as $3,760,000, $48,200, and $6,500 for fatal, injury, and property damage crashes, respectively. This provided an average weighted crash cost of $43,533. This average cost was multiplied by the number of crashes per mile to compute the cost associated with different access spacing scenarios. These costs can assist policy makers in quantifying the trade-offs of different access management regulations.

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NOTICE

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EXECUTIVE SUMMARY

The adequate spacing and design of access to crossroads in the vicinity of freeway ramps is critical to the safety and traffic operations of both the freeway and the crossroad. The research presented in this report develops a methodology to evaluate the safety impact of different access road spacing standards. The results clearly demonstrate the shortcomings of the AASHTO standards and the benefits of enhancing these standards. The models developed as part of this research effort were utilized to compute the crash rate associated with alternative section spacing. The study demonstrates that the models satisfied the statistical requirements and provide reasonable crash estimates. The results demonstrate an eight-fold decrease in the crash rate when the access road spacing increases from 0 to 300 m. An increase in the minimum spacing from 90 m (300 ft) to 180 m (600 ft) results in a 50% reduction in the crash rate. The models were used to develop lookup tables that quantify the impact of access road spacing on the expected number of crashes per unit distance, as illustrated in Table ES-1. The table demonstrates a decrease in the crash rate as the access road spacing increases. The expected number of crashes for access road spacings of 100 and 300 ft highlight the current criteria for access road spacing.

Table ES-1. Variation in the Expected Number of Yearly Crashes per Mile As a Function of the Access Section Length and AADT

L (m)	L (ft)	AADT (veh/day)										
		5000	10000	15000	20000	25000	30000	35000	40000	45000	50000	75000
0.0	0											
15.3	50	19.98	36.28	51.42	65.86	79.80	93.35	106.59	119.57	132.32	144.88	205.35
30.6	100	17.99	32.67	46.30	59.31	71.86	84.06	95.99	107.67	119.15	130.46	184.92
45.9	150	16.20	29.42	41.69	53.40	64.71	75.70	86.43	96.95	107.29	117.47	166.51
61.2	200	14.59	26.49	37.55	48.09	58.27	68.16	77.83	87.30	96.62	105.78	149.94
76.5	250	13.14	23.85	33.81	43.30	52.47	61.38	70.08	78.62	87.00	95.25	135.02
91.7	300	11.83	21.48	30.44	38.99	47.25	55.27	63.11	70.79	78.34	85.77	121.58
107.0	350	10.65	19.34	27.41	35.11	42.54	49.77	56.83	63.75	70.54	77.24	109.48
122.3	400	9.59	17.42	24.69	31.62	38.31	44.82	51.17	57.40	63.52	69.55	98.58
137.6	450	8.64	15.68	22.23	28.47	34.50	40.36	46.08	51.69	57.20	62.63	88.77
152.9	500	7.78	14.12	20.02	25.64	31.06	36.34	41.49	46.54	51.51	56.39	79.94
168.2	550	7.00	12.72	18.02	23.09	27.97	32.72	37.36	41.91	46.38	50.78	71.98
183.5	600	6.31	11.45	16.23	20.79	25.19	29.47	33.64	37.74	41.77	45.73	64.82
198.8	650	5.68	10.31	14.61	18.72	22.68	26.53	30.30	33.98	37.61	41.18	58.37
214.1	700	5.11	9.28	13.16	16.86	20.42	23.89	27.28	30.60	33.87	37.08	52.56
229.4	750	4.61	8.36	11.85	15.18	18.39	21.51	24.57	27.56	30.49	33.39	47.33
244.6	800	4.15	7.53	10.67	13.67	16.56	19.37	22.12	24.81	27.46	30.07	42.62
259.9	850	3.73	6.78	9.61	12.31	14.91	17.44	19.92	22.34	24.73	27.07	38.37
275.2	900	3.36	6.10	8.65	11.08	13.43	15.71	17.94	20.12	22.27	24.38	34.55
290.5	950	3.03	5.50	7.79	9.98	12.09	14.15	16.15	18.12	20.05	21.95	31.12
305.8	1000	2.73	4.95	7.02	8.99	10.89	12.74	14.54	16.31	18.05	19.77	28.02
321.1	1050	2.46	4.46	6.32	8.09	9.80	11.47	13.10	14.69	16.26	17.80	25.23
336.4	1100	2.21	4.01	5.69	7.29	8.83	10.33	11.79	13.23	14.64	16.03	22.72
351.7	1150	1.99	3.61	5.12	6.56	7.95	9.30	10.62	11.91	13.18	14.43	20.46
367.0	1200	1.79	3.25	4.61	5.91	7.16	8.37	9.56	10.73	11.87	13.00	18.42
382.3	1250	1.61	2.93	4.15	5.32	6.45	7.54	8.61	9.66	10.69	11.70	16.59
397.5	1300	1.45	2.64	3.74	4.79	5.80	6.79	7.75	8.70	9.63	10.54	14.94
412.8	1350	1.31	2.38	3.37	4.31	5.23	6.11	6.98	7.83	8.67	9.49	13.45
428.1	1400	1.18	2.14	3.03	3.88	4.71	5.51	6.29	7.05	7.80	8.55	12.11
443.4	1450	1.06	1.93	2.73	3.50	4.24	4.96	5.66	6.35	7.03	7.69	10.91
458.7	1500	0.96	1.73	2.46	3.15	3.82	4.46	5.10	5.72	6.33	6.93	9.82

An attempt was made to quantify the safety cost of alternative access road spacing using a weighted average crash cost. The weighted average crash cost was computed considering that 0.6, 34.8, and 64.6% of the crashes were fatal, injury, and property damage crashes, respectively. These proportions were generated from the field observed data. The cost of each of these crashes was provided by VDOT as $3,760,000, $48,200, and $6,500 for fatal, injury, and property damage crashes, respectively. This provided an average weighted crash cost of $43,533. This average cost was multiplied by the number of crashes per mile to compute the cost associated with different access spacing scenarios, as summarized in Table ES-2. It is anticipated that Table ES-2 can assist policy makers in quantifying the trade-offs of different access management regulations.

Table ES-2. Cost of Crashes Considering Crashes per Mile (Million Dollars)

L (m)	L (ft)	AADT (veh/day)										
		5000	10000	15000	20000	25000	30000	35000	40000	45000	50000	75000
0.0	0											
15.3	50	0.87	1.58	2.24	2.87	3.47	4.06	4.64	5.21	5.76	6.31	8.94
30.6	100	0.78	1.42	2.02	2.58	3.13	3.66	4.18	4.69	5.19	5.68	8.05
45.9	150	0.71	1.28	1.82	2.32	2.82	3.30	3.76	4.22	4.67	5.11	7.25
61.2	200	0.64	1.15	1.63	2.09	2.54	2.97	3.39	3.80	4.21	4.61	6.53
76.5	250	0.57	1.04	1.47	1.89	2.28	2.67	3.05	3.42	3.79	4.15	5.88
91.7	300	0.52	0.93	1.33	1.70	2.06	2.41	2.75	3.08	3.41	3.73	5.29
107.0	350	0.46	0.84	1.19	1.53	1.85	2.17	2.47	2.78	3.07	3.36	4.77
122.3	400	0.42	0.76	1.07	1.38	1.67	1.95	2.23	2.50	2.77	3.03	4.29
137.6	450	0.38	0.68	0.97	1.24	1.50	1.76	2.01	2.25	2.49	2.73	3.86
152.9	500	0.34	0.61	0.87	1.12	1.35	1.58	1.81	2.03	2.24	2.46	3.48
168.2	550	0.30	0.55	0.78	1.00	1.22	1.42	1.63	1.82	2.02	2.21	3.13
183.5	600	0.27	0.50	0.71	0.90	1.10	1.28	1.46	1.64	1.82	1.99	2.82
198.8	650	0.25	0.45	0.64	0.81	0.99	1.16	1.32	1.48	1.64	1.79	2.54
214.1	700	0.22	0.40	0.57	0.73	0.89	1.04	1.19	1.33	1.47	1.61	2.29
229.4	750	0.20	0.36	0.52	0.66	0.80	0.94	1.07	1.20	1.33	1.45	2.06
244.6	800	0.18	0.33	0.46	0.59	0.72	0.84	0.96	1.08	1.20	1.31	1.86
259.9	850	0.16	0.30	0.42	0.54	0.65	0.76	0.87	0.97	1.08	1.18	1.67
275.2	900	0.15	0.27	0.38	0.48	0.58	0.68	0.78	0.88	0.97	1.06	1.50
290.5	950	0.13	0.24	0.34	0.43	0.53	0.62	0.70	0.79	0.87	0.96	1.35
305.8	1000	0.12	0.22	0.31	0.39	0.47	0.55	0.63	0.71	0.79	0.86	1.22
321.1	1050	0.11	0.19	0.28	0.35	0.43	0.50	0.57	0.64	0.71	0.77	1.10
336.4	1100	0.10	0.17	0.25	0.32	0.38	0.45	0.51	0.58	0.64	0.70	0.99
351.7	1150	0.09	0.16	0.22	0.29	0.35	0.40	0.46	0.52	0.57	0.63	0.89
367.0	1200	0.08	0.14	0.20	0.26	0.31	0.36	0.42	0.47	0.52	0.57	0.80
382.3	1250	0.07	0.13	0.18	0.23	0.28	0.33	0.37	0.42	0.47	0.51	0.72
397.5	1300	0.06	0.11	0.16	0.21	0.25	0.30	0.34	0.38	0.42	0.46	0.65
412.8	1350	0.06	0.10	0.15	0.19	0.23	0.27	0.30	0.34	0.38	0.41	0.59
428.1	1400	0.05	0.09	0.13	0.17	0.20	0.24	0.27	0.31	0.34	0.37	0.53
443.4	1450	0.05	0.08	0.12	0.15	0.18	0.22	0.25	0.28	0.31	0.33	0.47
458.7	1500	0.04	0.08	0.11	0.14	0.17	0.19	0.22	0.25	0.28	0.30	0.43

INTRODUCTION

The current Access Control Agreement for Interstate Interchanges in Virginia between the Virginia Department of Transportation (VDOT) and the Federal Highway Administration (FHWA), dated April 28, 2004, sets the minimum length of access control on crossroads as 100 ft in urban areas and 300 ft in rural areas, as measured from the terminal of the ramp. The Department frequently receives requests from developers to break this agreement and allow commercial entrances to be placed at shorter distances. VDOT staff can spend considerable time reviewing the traffic, safety, and operational impacts of these access points. Research indicates that effective planning and access management can ultimately help, instead of hinder, the development potential of interchange areas (AASTHO Roadside Design Guide, Washington, DC, 2002). Research is needed to determine if the above minimum lengths are optimal.

PURPOSE AND SCOPE

While several research studies have been conducted to assess the safety impacts of different access management techniques (including traffic signal spacing, median alternatives, etc.), research is needed to quantify the safety impact associated with access control spacing along arterial crossroads in the vicinity of freeway interchanges. Consequently, the research presented in this report develops regression models that relate crash frequencies to the geometric and traffic variables associated with access roads in the vicinity of freeway interchanges.

The objectives of this report are threefold. First, the report presents a synthesis of the state-of-practice access management regulations in various states within the United States with regard to interchange access control. Second, the research investigates the safety impact of varying access types and lengths using data assembled as part of the study. Third, the study develops regression models that relate crashes to various roadway and geometric variables. These models are intended to assist policy makers in setting access management standards.

METHODS

The procedures followed during the research were:

1. Review the literature.
2. Investigate how the American Association of State Highway and Transportation Officials (AASHTO) procedures arrived at the 100 ft and 300 ft recommendations.
3. Identify and characterize candidate field locations for the study.
4. Analyze crash databases to identify the safety impacts of alternative access road spacing.
5. Compare the frequency of crashes and crash rate (crashes/vehicle volume) for different access road spacing over a period of five years to identify the safety hazard of short access distances
6. Develop models that relate crashes to access spacing.
7. Prepare a report documenting the results of the field crash analysis.

The entire research procedure for this project is summarized in Figure 1. This procedure is divided into three stages. In Stage 1 data collection is performed, in Stage 2 a qualitative data analysis is performed, and in Stage 3 crash prediction models are developed.

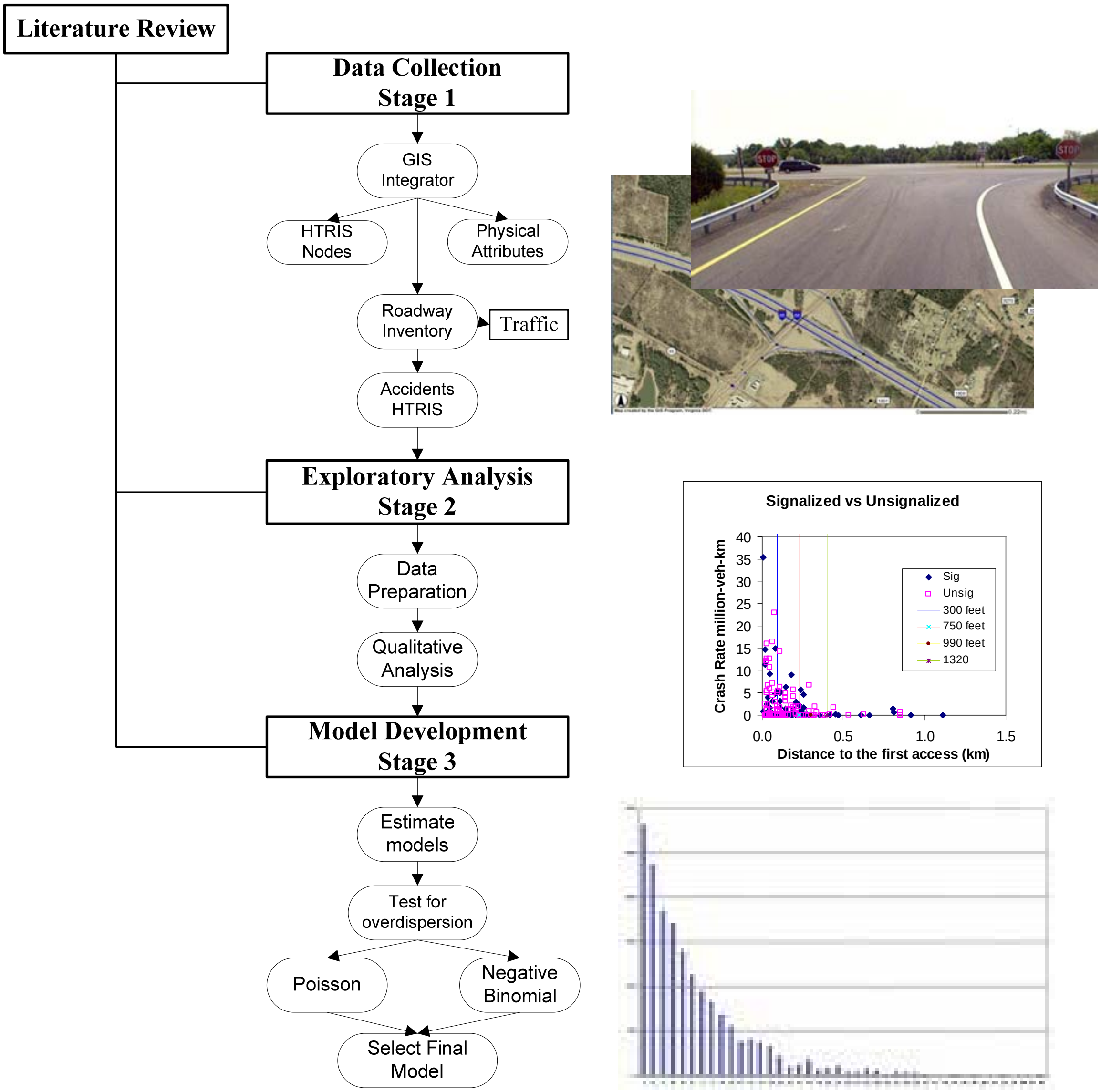


Figure 1. Flowchart of Research Procedure

RESULTS

Literature Review

The team conducted an extensive literature review that included published and unpublished, domestic and international material as well as practical applications and field experiences of access management in the vicinity of interchanges. Major references in access management that were referenced include the AASHTO policies, the TRB Access Management Manual, NCHRP reports, DOT and national standards, and access management conference proceedings. This section provides a brief summary of the literature review.

Factors Influencing Access Location Spacing

Several factors affect the distances downstream of an interchange terminal at which an access location can be permitted. NCHRP Report 332 *Access Management on Crossroads in the Vicinity of Interchanges* identifies the following factors:

- Surrounding land use and environment
- Roadway classification
- Interchange form
- Public and private accesses
- Type of downstream access point
- Downstream storage requirements
- Cross section
- Design speed
- Volume
- Signal cycle length
- Cost and economic impacts
- Level of interchange importance
- Crossroad jurisdiction.

Surrounding Land Use and Environment

In general, more strict access standards are required at rural interchanges because the land use density is much lower, parcel size is much larger, and usually there is a higher posted speed limit that requires longer distances to decelerate or to turn to avoid a conflict maneuver. On the contrary, urban interchanges cater to smaller parcel sizes and, in general, have lower speed limits.

Roadway Classification

Higher classified roadways (freeways, expressways and arterials) require longer access spacing than lower classified roads.

Interchange Form

From an access spacing standpoint, interchanges can be categorized as those with free-flowing entrances and exit ramps and those where ramp entrances and terminals are controlled by traffic signals or stop signs. For the first type, the access distance must consider: the merge where the ramp traffic enters the arterial, the weaving movements to enter the median lanes, the transition into left-turn lanes, and the required storage length. The signalized ramp intersection is treated similarly to other signalized intersections; however, queuing from the ramp onto the freeway mainline must be avoided.

Public and Private Accesses

Public access points are more restricted (greater access distances are required) because they generate more traffic compared with traffic at private accesses, increasing the number of potential conflicts.

Type of Downstream Access Point

Several agencies do not differentiate between either the type of turning movement or traffic control associated with the access point on the crossroad. In addition, they specify standards for the first access independent of the type of access.

Other agencies will allow private or public approaches that are controlled to right-in only access, right-out only access, or right-in/right-out access movements downstream of the interchange terminal to be closer than those access points that allow a left-turn or required a weaving maneuvers. Unsignalized and signalized full-access intersections on crossroads are generally recommended to be located at the farthest point downstream of the interchange terminal.

The access space required to make the decisions and complete the vehicular maneuver is usually increased in the following order:

- Right –In Only Access
- Right-Out Only Access
- Right-In/Right-Out Access
- Left-In/Right-In/Right-Out Access
- Unsignalized, Full Access
- Signalized, Full Access.

Downstream Storage Requirements

Very few agencies required that the downstream vehicular queue storage be considered in determining access spacing.   The downstream storage requirements are based on vehicular demand, cycle length and intersection geometry. Because this process depends on the knowledge of future traffic demands, the storage required takes into account the specific land use and the street network in the vicinity of the interchange under both near and long-term conditions.

Cross Section

Some agencies consider the cross section of the cross road when determining the proper access spacing. Several states examine the interchange form to determine if weaving maneuvers will occur. The spacing distance can be determined by the number of lanes that need to be crossed, the advancing volume on the cross roads and left turn storage requirements at the downstream intersection.

Design Speed

The design speed affects the stopping sight distance and the decision sight distance. The higher the design speed, the longer the distance required.

Volume

The volume on the crossroad can be incorporated directly or indirectly (as road classification) as one of the components of access distance.

Traffic Signal Cycle Length

To assure adequate progression of vehicles and maintain minimum queues between the downstream traffic signal and the ramp terminal, some states use signal cycle length to determine the access spacing on the crossroads. The shorter the cycle length for the signal, the shorter the access distance required.

Cost and Economic Impacts

Some agencies consider the level of importance of the interchange. The higher the importance of the main line and the crossroad facilities the stricter (greater distances or not permitted) the standards are set.

Crossroad Jurisdiction

The jurisdiction of the crossroad can dictate the access management separation. Some local jurisdictions have different standards than the state DOTs and in these cases the access distance is defined by those who have jurisdiction of the road.

Considerations Regarding the Type of First Access

When a driver exits a freeway and enters a crossroad, the driver must be able to perceive and react to unexpected and unusual conditions. The operations on crossroads in the vicinity of an interchange are often more complex than the operations through the rest of the road/street system. The drivers are exiting or entering a facility that is higher speed, and access controlled to or from a crossroads that has entrances and exits in many different configurations. The different maneuvers that the drivers need to accomplish depend upon factors such as the type of road and the type of first access. The Oregon Department of Transportation sponsored one of the most complete studies regarding interchange access spacing (Layton, 1996), which will be discussed in some detail in the following sections.

According to the Oregon study, when multilane roads are considered, the spacing to the nearest major intersection is the weaving distance plus the queue length at the intersection. The weaving maneuvers must be completed by the time the end of the queue at the intersection is reached. The distance must also be sufficient to assure vehicles have adequate distance to weave comfortably to the leftmost lane before being trapped in the right lane by vehicles queuing back from the intersection.

For the location of the first driveway, three criteria must be considered: stopping sight distance, minimum access spacing to maximum egress capacity, and decision sight distance. The stopping sight distance must be taken into account because the driver must have enough distance to see operations and vehicles at the next driveway with enough distance to stop. The maximum egress capacity will improve the traffic stream characteristics (see below). The third criterion, decision sight distance, is important because the driver must have enough time to react to unusual situations and this distance allows for an increase in perception-reaction time with increasingly complex traffic operations.

In the case of two-lane roads the drivers exiting from the freeway have higher level of expectations than the ones on the crossroads. Several elements including the complexity of the interchange area, uniqueness of the operations and mix of drivers requires more time for drivers to perceive and react properly. Consequently, to guarantee smooth operations and safety, the decision sight distance must be provided. The second major factor for this type of road is the queuing distance required to accommodate all the vehicles waiting to enter the nearest intersection. The location of the first right and the first left follows the same regulations as for the first right in the case of multilane roads.

The following section describes the different distances that must be taken into account when computing access separation distances.

Stopping Sight Distance

Stopping Sight Distance is the distance necessary to come to a complete stop to avoid collision with another vehicle that is decelerating or stopping while turning at an access point. According to the AASHTO Green Book, the stopping sight distance is the sum of two distances: (1) the distance traversed by the vehicle from the instant the driver sees an object necessitating a stop to the instant the brakes are applied (known as brake reaction distance), and (2) the distance needed to stop the vehicle from the instant the brake is applied (known as braking distance) and can be computed as:

$$D = 1.47\ vt + 1.075\ (V^2/a) \qquad [1]$$

where
D = stopping sight distance (ft),
v = design speed (mph),
t = braking reaction time (2.5 s), and

a = deceleration rate (ft/s^2).

Table 1 shows the stopping sight distance for a breaking reaction time of 2.5 sec and a deceleration rate of 3.4 m/s^2.

Table 1. Stopping Sight Distance Computation

Metric					US Customary				
Design Speed (Km/h)	Brake reaction distance	Breaking distance on level	Stopping Sight Distance Calculated (m)	Design (m)	Design Speed (mph)	Brake reaction distance	Breaking distance on level	Stopping Sight Distance Calculated (ft)	Design (ft)
20	13.9	4.6	18.5	20	15	55.1	21.6	76.7	80
30	20.9	10.3	31.2	35	20	73.5	38.4	111.9	115
40	27.8	18.4	46.2	50	25	91.9	60	151.9	155
50	34.8	28.7	63.5	65	30	110.3	86.4	196.7	200
60	41.7	41.3	83	85	35	128.6	117.6	246.2	250
70	48.7	56.2	104.9	105	40	147	153.6	300.6	305
80	55.6	73.4	129	130	45	165.4	194.4	359.8	360
90	62.6	92.9	155.5	160	50	183.8	240	423.8	425
100	69.5	114.7	184.2	185	55	202.1	290.3	492.4	495
110	76.5	138.8	215.3	220	60	220.5	345.3	566	570
120	83.4	165.2	248.6	250	65	238.9	405.3	644.4	645
130	90.4	193.8	284.2	285	70	257.3	470.3	727.6	730
					75	275.6	539.9	815.5	830
					80	294	614.3	908.3	910

Note: Brake reaction distance predicated on a time of 2.5 s; deceleration rate of3.4m/s^2(11.2ft/s^2) used to determine calculated sight distance

From *A Policy on Geometric Design of Highways and Streets*, 2004, by the American Association of State Highway and Transportation Officials, Washington, D.C. Used by permission.

Decision Sight Distance

Stopping sight distances are sufficient when the drivers do not need to make complex or instantaneous decisions but are not appropriate for situations when information is difficult to perceive, or when unexpected or unusual maneuvers are required. Decision sight distance is the distance necessary to perceive and react to unexpected, unusual, or complex conditions (speed/ path/ direction change) allowing reasonable competent drivers to come to a hurried stop under ordinary circumstances.

$$D = 1.47\, vt \qquad [2]$$

where
D = decision sight distance (ft),
v = design speed (mph), and
t = total pre-maneuver and maneuver time (10.2 to14.5 s).

Weaving Distances

Most weaving analyses have focused on freeway operations and the majority of the standards have based their weaving distances calculation on Jack Leisch curves (Layton, 1996; Leisch, 1982) as illustrated in Figure 2.

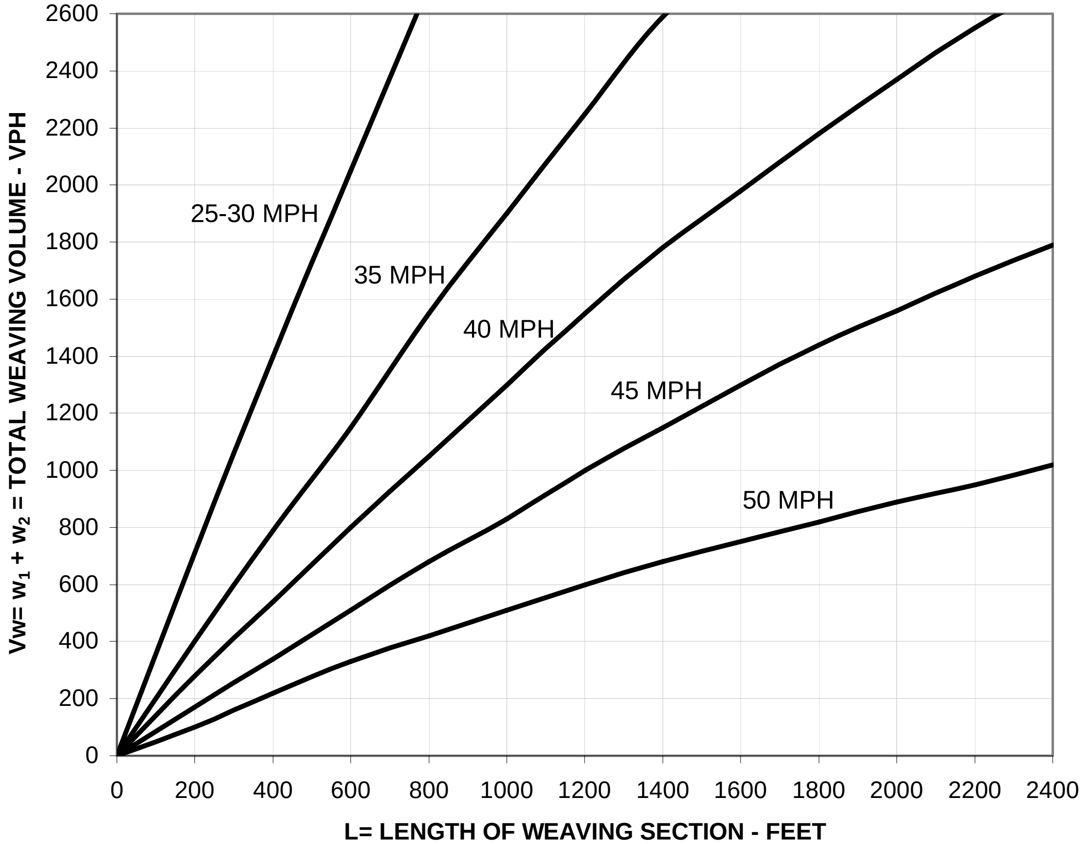


Figure 2. Analysis of Service Road Weaving Conditions (Layton, 1996)

Table 2 shows the weaving distances for different weaving volumes and speeds. According to Leisch, for normal conditions, weaving distances of 700 ft to 800 ft are required for two-lane roads and 1200 ft to 1600 ft will usually be adequate for multilane roads.

Table 2. Estimated Weaving Distances

Weaving Volume (vph)	Speed (mph) 25-30	35	40	45	50
200	50	100	150	200	400
400	100	200	300	450	800
600	150	300	450	700	1200
800	250	400	600	950	1800
1000	300	500	750	1200	2400
1200	350	600	900	1450	
1400	400	710	1050	1700	
1600	450	820	1200	2050	
1800	500	930	1400	2400	
2000	600	1040	1600		
2200	700	1150	1800		
2400	800	1270	2050		
2600	900	1400	2300		

Source: Gluck, J., H.S. Levinson, and V. Stover. NCHRP Report 420: Impacts of Access Management Techniques, Transportation Research Board, Washington, DC, 1999.

Transition Distance

The transition or "lane change" distance to enter the storage lane depends on the approach speed and the number of lanes to be crossed and varies between 200 and 300 ft.

Left-Turn Storage

Left-turn storage lanes should be adequate to handle the anticipated turning volumes with a low likelihood of overflow or failure. Storage length can be estimated from the following equation:

$$L = 25 \times \frac{RV}{N_c} = 25 \times Rl \qquad [3]$$

where
V = Left turns per hour (flow rate)
N_c = Cycles per hour
l = Left turns per cycle
R = Randomness factor for less than 5 percent failure. R = 2.0 for random operations (i.e., rural); R = 1.5 for operations where traffic tends to platoon.
L = Length of left-turn storage in feet

Where there are dual left-turn lanes, the resulting value can be reduced by roughly 45%. Thus, the length of a single left-turn lane in feet may be estimated to be as much as 50 times the number of left turns per cycle. For dual left-turn lanes, the length of each lane in feet may be estimated to be as much as 28 times the number of left turns per cycle. The actual storage will also depend on the degree of randomness of arriving left-turning vehicles.

Street Width Distance

Where separation distances are measured from the centerline of the road crossing the arterial, an additional distance of ½ the width of the Right of Way must be added to the required distance to compute the minimum access spacing as shown in Figure 5.

Perception-Reaction Distance

This distance must be considered when the driver faces unfamiliar situations calculated at 2.5 ft/s; it represents an additional 125 ft.

Maximum Egress Capacity

Maximum egress capacity is the distance necessary for vehicles to accelerate from 0 to the through traffic speed multiplied by 1.5. Major and Buckley (1962) found that driveways spaced at distances greater than the distance necessary to accelerate from 0 to the through traffic speed multiplied by 1.5 reduces delay to vehicles entering the traffic stream and improves the rate in which vehicles can merge into the traffic stream.

Table 3. Required Distance for Maximum Egress Capacity

Type of area	Speed (mph)	1 X Acc Distance (ft)	1.5 X Acc Distance (ft)	M
Urban	35	300	450	990
Suburban/Urban	45	575	860	1320
Rural	55	1000	1500	1320

How the Distance Is Measured

There are numerous ways to define access spacing distances from the off-ramp of an interchange which can be visualized by considering Figure 3.

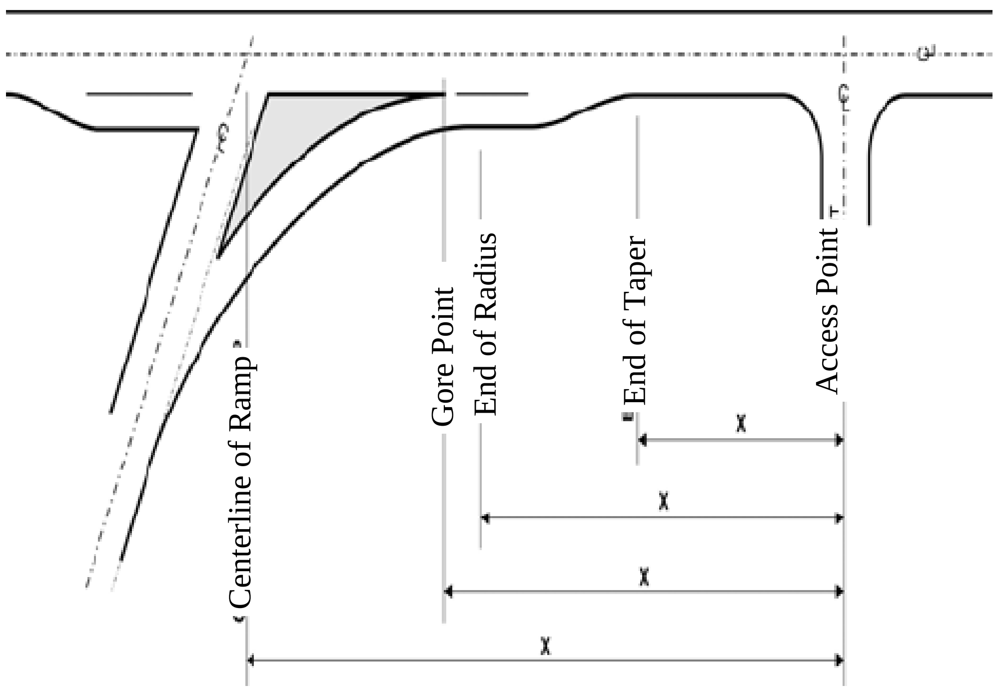


Figure 3. Different Ways That the Access Space Distance Can Be Measured

Source: Butorac, M., and J. Wen. Access Management on Crossroads in the Vicinity of Interchanges, A Synthesis of Highway Practice. *Transportation Research Board*, Washington, DC, 2004.

- *Centerline-to-Centerline*—longitudinal distance between the geometric intersections of the off-ramp or on-ramp centerline with the centerline of the cross-road, and the downstream access point centerline with the centerline of the crossroad.
- *Gore Point*—point at which the off-ramp's inside edge of pavement and the crossroad's outside edge of pavement merges.
- *End of Radius* (*tangent section*)—point at which the radial edge of pavement or curb transition between the off-ramp and crossroad terminates or becomes parallel to the crossroad centerline.
- *End of Taper*—point at which the off-ramp acceleration lane and the crossroad outside lane merge.

National Standards

There are basically four national documents that are used by the states in access management separation and control standards, which include:

- AASHTO Policy on Design Standards – Interstate System (2005)
- AASHTO Policy on Geometric Design of Highways and Streets (2004)
- NCHRP Report 420 Impacts of Access Management Techniques (1992)
- Access Management Manual (2003)

Each of these documents is briefly described in this section.

AASHTO Policy on Design Standards – Interstate System (2005)

The AASHTO booklet, *A Policy on Design Standards – Interstate System* (prepared by the Standing Committee on Highways, AASHTO Highway Subcommittee on Design and the Technical Committee on Geometric Design) states that: "Access control shall extend the full length of ramps and terminals on the crossroads. Such control shall either be acquired outright prior to construction or by the construction of frontage roads or by the combination of both. Access control beyond the ramp terminals should be affected by purchasing access rights, providing frontage roads, controlling added corner right of way areas, or prohibiting driveways. Such control should extend beyond the ramp terminal at least 30 m (100 ft) in urban areas and 90 m (300 ft) in rural areas. However, in areas of high traffic volumes, where there exists the potential for development which would create operational or safety problems, longer lengths of access control should be provided."

However, no underlying rationale is presented in the document for these recommended minimum distances of 100 ft and 300 feet, which are currently used by VDOT (see later section on derivation of these minimums).

AASHTO Policy on Geometric Design of Highways and Streets (2004)

According to AASHTO's *A Policy on Geometric Design of Highways and Streets* (Green Book), "the appropriate degree of access control or access management depends on the type and importance of an arterial. Anticipation of future land use is a critical factor in determining the degree of access control. Provision of access management is vital to the concept of an arterial route if it is to provide the service life for which it is designed." The objectives of the access separation distance are to minimize spillback on the ramp and crossroad approaches to the ramp terminal, and to provide adequate distance or space for crossroad weaving, merging maneuvers and for storage of turning vehicles at access connections on the crossroad. The document classified the distance standards according to the type of interchange. As seen in Figure 4, at an interchange with free-flow ramps entering and exiting from a crossroad, the preferred access control distance includes:

- The distance it takes a vehicle to merge from the ramp into the outside lane on the crossroad
- Distance a vehicle needs to merge into the inside lane
- Distance required for a vehicle to move into a left-turn lane
- The length of storage required for the left-turn lane.

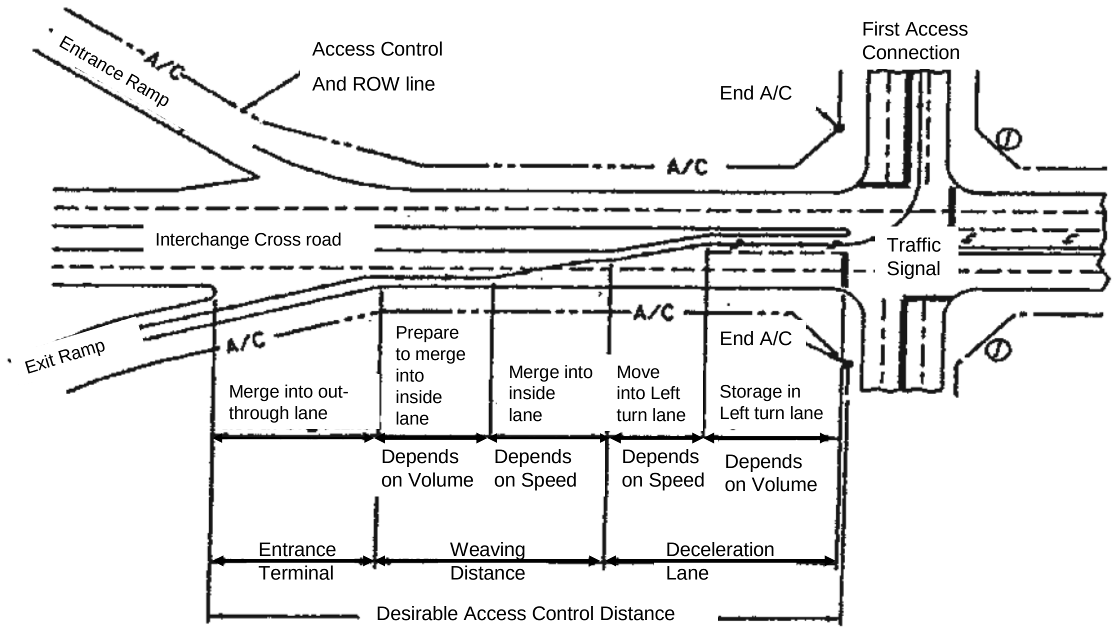


Figure 4. Free Flow Ramps Entering and Exiting From Cross Road

From *A Policy on Geometric Design of Highways and Streets*, 2004, by the American Association of State Highway and Transportation Officials, Washington, D.C. Used by permission.

For diamond interchanges or other interchange forms without free-flow ramps (the first access may either be controlled by a traffic signal or stop signs), the desirable access control distance on the crossroad includes:

- Distance required for advance guide signs, progression
- Storage lengths of traffic turning at the first access.

NCHRP Report 420 Impacts of Access Management Techniques (1992)

The NCHRP Report 420 defines the separation distances slightly differently than AASHTO, as shown in Figure 5 and Table 4.

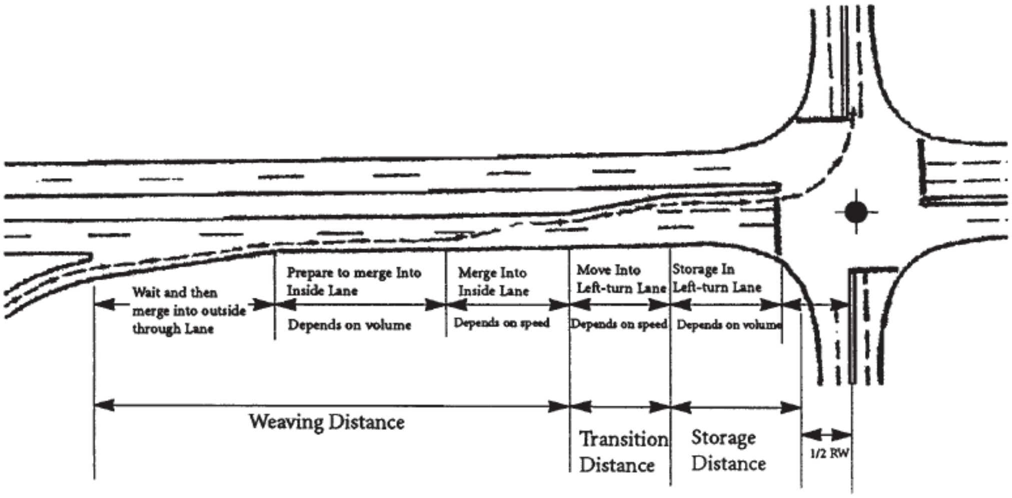


Figure 5. Access Separation Distance

Source: Gluck, J., H.S. Levinson, and V. Stover. NCHRP Report 420: Impacts of Access Management Techniques, Transportation Research Board, Washington, DC, 1999.

The preferred access control distance includes:

- Distance required to weave across the through travel lanes
- Distance required for transition into the left-turn lane or lanes
- Distance needed to store left turns with a low likelihood of failure
- Distance from the stop line to the centerline of the intersecting road or driveway
- Distance covered during driver perception-reaction time (could be added)

Table 4. Components of Access Control Distance

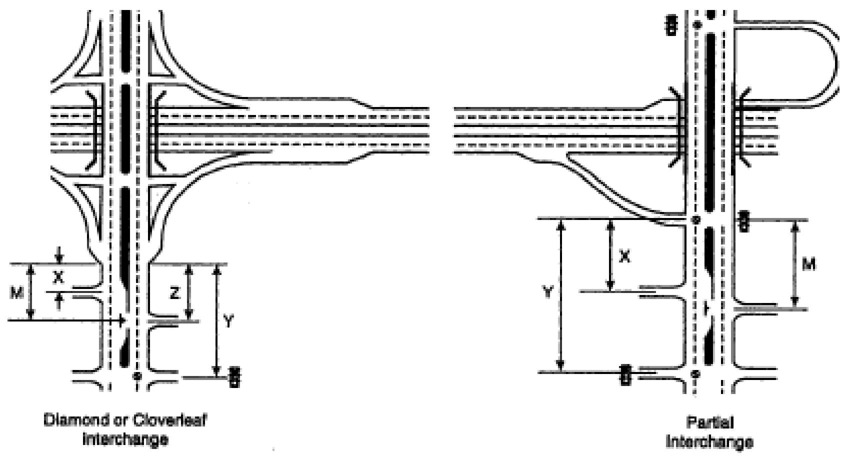
Source: Gluck, J., H.S. Levinson, and V. Stover. NCHRP Report 420: Impacts of Access Management Techniques, Transportation Research Board, Washington, DC, 1999.

Component of Access Control Distance	Recommended Access Spacing
Perception–Reaction Distance	25 ft
Lane Transition	50–250 ft
Left Turn Storage	Estimate using equation or use 50 ft per left turn per cycle
Weaving Distance	700 to 800 ft, two-lane arterials 1,200 to 1,600 ft, multilane arterials
Distance to Centerline of Cross Street	50ft

Access Management Manual

The *Access Management Manual* is a well-known Transportation Research Board (TRB) publication by the Committee on Access Management, which summarizes the state of the art in access management and provides technical information on access management techniques, drawing from the knowledge of many experienced practitioners. Chapter 9, Access Spacing includes a section on Interchange Area Management where it recommends guidelines based on

NCHRP report 420 and research conducted by Lytton (see DOT standards). The Manual includes guidelines for access spacing ranging from 750 ft (230 m) to ½ mile (800 m). Minimum spacing standards are presented for multilane crossroads and two lane roads for diamond or cloverleaf interchanges and for partial interchange as shown in Figure 6 and Figure 7.

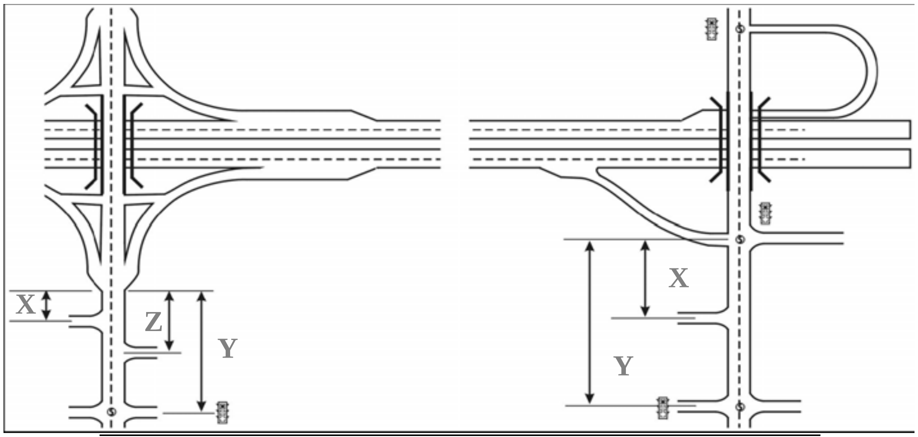


Type of Area	**Spacing Dimension**			
	X	Y	Z	M
Fully Developed	750 ft	2640 ft	990 ft	990 ft
Urban	(230 m)	(800 m)	(300 m)	(300 m)
Suburban/Urban	990 ft	2640 ft	1320 ft	1320 ft
	(300 m)	(800 m)	(400 m)	(400 m)
Rural	1320 ft	2640 ft	1320 ft	1320 ft
	(400 m)	(800 m)	(400 m)	(400 m)

X = distance to first approach on the right; right-in/right-out only.
Y = distance to first major intersection. No four legged intersections may be placed between ramp terminals and the first major intersection
Z = distance between the last access connection and the start of the taper for the on-ramp.
M = distance to the first directional median opening. No full median openings are allowed in non-traversable medians up to the first major intersection.

Figure 6. Minimum Spacing for Freeway Interchange Areas for Multilane Roads.

Source: Access Management Manual, Committee on Access Management, Transportation Research Board, Washington, DC, 2003.



Type of Area	Spacing Dimension	
	X or Z	Y
Fully Developed Urban	750 ft (230 m)	1320 ft (400 m)
Suburban/Urban	990 ft (300 m)	1320 ft (400 m)
Rural	1320 ft (400 m)	1320 ft (400 m)

Figure 7. Minimum Spacing for Freeway Interchange Area for Two-Lane Road
Source: Access Management Manual, Committee on Access Management, Transportation Research Board, Washington, DC, 2003.

VDOT Standards

VDOT currently does not have an official access management manual aside from the minimal standards and regulations contained in the "Minimum Standards of Entrances to State Highways." In 2007 the General Assembly directed VDOT to develop and implement access management standards and regulations to be in effect July 1, 2008.

The current Access Control Agreement for Interstate Interchanges in Virginia between VDOT and FHWA was signed on April 28, 2004. The purpose of the agreement was to clarify the design standards and implementation policy for access control at new and modified interchanges along interstates in Virginia. The agreement mandates that all new interchanges as well as existing interchanges with proposed improvements be evaluated for safety and operational needs to determine the appropriate lengths of access restrictions. In addition, the agreement mandates that VDOT make every effort to control access on the cross roads. Furthermore, the minimum length shall not be less than the following:

1. *Urban Areas (Census Determined Areas with a population of 50,000 or more)*: The AASHTO standard for the length of Control of Access should be as long as practical but shall be a minimum of 100 ft beyond the proposed ramp terminal.
2. *Rural Areas*: The AASHTO standard for length of Control of Access is a minimum of 300 ft beyond the proposed ramp terminal.
3. *Planned Urban Areas (population forecast of 50,000 or more)*: The AASHTO standard for length of Control of Access is a minimum of 100 ft beyond the proposed ramp terminal. In a situation where an access point exists less than 300 ft and over

> 100 ft from the ramp terminal, VDOT should make a concerted effort to relocate the access outside the 300 ft or as far from the proposed ramp terminal as possible. In a situation where an access point does not already exist, VDOT should acquire a minimum of 300 ft of controlled access beyond the ramp terminal.

The 2004 Agreement and a copy of a 1999 VDOT Memorandum on how these distances are measured are included in Appendix A.

How Did AASHTO Derive the 100 ft and 300 ft Recommendation?

Several state departments of transportation, including VDOT, rely on the spacing guidelines provided in the AASHTO publication, *A Policy on Design Standards – Interstate Systems (2005)*, when providing access control beyond the ramp terminals.

The research team conducted an exhaustive search and Robert Walters, Chairman of the Technical Committee in Geometric Design 2004, provided the following and best explanation.

The 1957 AASHTO *Guide for the Application of Frontage Roads on the National System of Interstate and Defense Highways* recognized that there may be an advantage in extending control of access along the crossroad for some distance beyond the ramp terminals . . . ensuring freedom of movement beyond the ramp terminals. A 1962 revision of this document retained this wording.

In 1963, AASHTO published *A Plan to Promote Proper Traffic Operation on Crossroads near Interchanges*. This plan appears to be an outgrowth of a report from AASHTO entitled *Report by the Special Freeway Study and Analysis Committee to the Executive Committee of the American Association of State Highway Officials, February 1960* published as an informational guide. The plan, in its discussion of the crossroad situation, states in part: "Concentration of roadside businesses and other industrial and commercial development within a few hundred to a few thousand feet of interchange ramp terminals results in a series of driveways and intersections that could greatly reduce the traffic-carrying ability and safety of the crossroad."

The plan states that one of the actions that can be taken is the extension of control of access: "Extension of Control of Access. This could promote free-flow conditions along the crossroad for the distance under access control. Steps should be taken to avoid congestion beyond the control of access limits." The plan recommends using all available authority to affect an extension of the control of access and where authority is lacking, take steps to obtain the necessary authority to provide working mechanisms.

Chapter VI of the 1965 AASHTO Blue Book (entitled Controlled Access Highways) makes the statement regarding access control at ramp terminals "one of which is the extension of control of access for a few hundred feet along the crossroad beyond the ramp terminal." The earliest resource available was primarily the 1954 AASHTO Blue Book. Traffic operations at ramp terminals could be likened somewhat to the merging maneuver at the end of a speed change lane (acceleration lane) where a taper is provided to allow the vehicles entering the traffic flow to safely merge with the through traffic. Taper lengths provided in the Blue Book ranged from 150 ft for low speed conditions to 300 ft for the higher speed conditions. Based on this information, it would be logical to use values in the 100 ft - 300 ft range. This would be consistent with the "a few hundred feet" statement in the 1965 Blue Book.

In 1966, a critical survey of the safety characteristics of the interstate and other highway systems was undertaken. The objective was to identify those aspects of design and operation in facilities in various sections of the country that could be improved to increase safety and the quality of traffic service. The report of this survey, *Highway Design and Operational Practices Related to Highway Safety* (Yellow Book) included a recommendation that "a 'zone' be established under complete control of the responsible highway or street department, desirably extending along the cross street beyond any ramp terminal. Through this zone, the cross street should have no commercial or private entrances and no signs other than those officially installed for safe and efficient operation of the highway." This recommendation led to a revision in the design standards for the interstate System.

The AASHTO adopted by letter ballot on October 15, 1966, certain changes in the "Geometric Design Standards for the National System of Interstate and Defense Highways" as the design standards were subtitled at that time. This became the fourth revision to the standards adopted July 12, 1956. The changes adopted were related to control of access. The revised standard reads: "On all sections of the Interstate System, access shall be controlled by acquiring access rights outright prior to construction or by the construction of frontage roads, or both. Control of access is required for all sections of the Interstate System, including the full length of ramps and terminals on the crossroad. Control for connections to the crossroad should be affected beyond the ramp terminals by purchasing of access rights, providing frontage roads to control access, controlling added corner right-of-way areas, or denying driveway permits. Such control should extend along the crossroads beyond the ramp terminal about 100 feet or more in urban areas and about 300 feet or more in rural areas." AASHTO subsequently reprinted the policy on design standards incorporating the changes in the control of access text.

The Bureau of Public Roads (BPR) (now FHWA) strongly endorsed the report by letter to chief administration officers of the state highway and transportation departments, and then through issuance of an Instructional Memorandum (IM 21-11-67, dated May 19, 1967). The IM made it bureau policy to incorporate provisions of the report in the plans for all projects for high-speed highways.

In 1974, the second edition of the Yellow Book was published. It updated and expanded the substance and coverage of the earlier publication. The second edition retained the discussion on access control on the crossroad at interchanges but modified it to acknowledge that urban and rural conditions may be different. The text added: "In urban areas, this control length may have to be limited to 100 feet."

The 1967 edition of the interstate standards was revised in December 1988, with the text reading "beyond the ramp terminal at least 100 feet in urban areas and 300 feet in rural areas." In the July 1991 update of the interstate standards, the text was expanded to add: "These distances should satisfy any congestion concerns. However, in areas where the potential for development exist which would create traffic problems, it may be appropriate to consider longer lengths of access control."

The 1997 edition of the Yellow Book provided a discussion that suggested increasing the distances from the ramp terminal to the cross road or prohibiting movements when weaving was significant.

In 2005 the new edition of the AASHTO Booklet *A Policy on Design Standards – Interstate System* slightly modified the wording of the previous version but retained the 100 and 300 feet distances.

State DOT Standards

The NCHRP Synthesis 332 research effort gathered information in regard to the current state of the practice in locating access points on crossroads in the vicinity of interchanges as:

- Nearly 90% of the surveyed state and provincial transportation agencies and toll authorities currently manage, to varying degrees, access to crossroad facilities upstream and downstream of the interchange terminals.
- Agencies use a wide range of factors to determine the appropriate spacing to the first access location downstream and upstream of the interchange terminal The majority of state departments of transportation rely on the 100-ft urban and 300-ft rural spacing guidelines provided by AASHTO.
- Access spacing standards for crossroad facilities vary in distance, from basically zero to 1,320 ft; however, only 50% of the transportation agencies with such standards had a specific methodology that was used to determine the actual distances.
- A variety of reference points are used by state agencies to determine the access spacing distance to the nearest downstream intersection on the crossroad.

Some agencies have developed their own methodology (Oregon, New Jersey, Florida) while several states rely on the spacing guidelines provided by AASHTO (Table 5). The spacing guidelines shown on Table 5 were compiled using information from NCHRP Synthesis 332 and state DOT websites. However, the research studies to determine access spacing are based on the operational analysis of the system and, although there are ways to evaluate quantitatively the operational benefits of different access spacing standards, a methodology for estimating the safety impacts has not been established.

Table 5. DOT Access Spacing Standards

	Right in/right out		Right in/right out- left in/left out		Signalized Intersection	
	Rural	**Urban**	**Rural**	**Urban**	**Rural**	**Urban**
Alberta Transportation	400m	200 m	400m	200m	400m	200m
Alabama			1320 ft(55mph)	750ft(45mph)	2640 ft	2640 ft
Ariz TRC	300ft		300ft			
Arkansas	300ft	150ft	300ft	150ft	300ft	
California DOT	125m minimum	160m prefered	125m minimum	160m prefered	125m minimum	160m prefered
Colorado DOT	350ft minimum	550ft prefered	350ft minimum	550ft prefered	1/4 mile likely , 1/2 mile desirable	
Delaware	Driveways provided in accordance to the AASHTO standards , no driveways withing 10 ft of an intersection radius , acceleration lane , decceleration lane					
Florida DOT	660ft (>45mph)	440ft (<45mph)	660ft (>45)	440ft (<45)	1320ft	
Georgia DOT	300ft	100ft	1320ft	1000ft	1320ft	660 ft
Idaho	1000ft	300 ft	1000ft	300 ft	0.5mile	0.25 mile
Illinois DOT	300ft	100ft	Varies as per the design speed, interchange type and ramp type			
Indiana DOT	300-500ft	100-200ft	300-500ft	100-200ft		
Iowa DOT	600ft	150-300ft			maximum possible	
Kansas DOT	None		None		1320ft	
Kentucky			R1 : 300ft , R2 : 300ft , R3: 150ft , R4 : 150 ft	U1: 300ft , U2 : 150ft , U3 150 ft , U4 : 100ft ,	R1: 1200ft , R2: 600ft , R3 : 450ft , R4 : 150ft	U1 : 1200ft , U2 :600ft , U3 :300ft , U4: 150ft
Louisiana DOT	300ft	100ft			300ft	100ft
Maine DOT	500ft for off ramps		500ft for off ramps		Depends on speed limit	
Maryland	100 -150ft				500ft	
Massachusetts	no access till 500 ft from the ramp					
Michigan DOT	300ft	100ft	300ft	100ft	300ft	100ft
Minnesota DOT	500-800ft		500-800ft		500-800ft	
Mississippi DOT	300ft	100ft	300ft	100ft	300ft	100ft
Missouri			major : 750-1320 ft / minor : NA		1320- 2640 ft	
Nebraska DOT	660ft minimum		660ft minimum			
Nevada DOT	300ft minimum	550ft prefered	300ft minimum	550ft prefered	1/2 mile between Signals	1/4 mile Int to first signal
New Brunswick DOT	65m minimum				65m minimum	
New Jersey DOT	300ft	100 ft	300 ft	100 ft	1/2 mile depending on speeed and cycle length	
New York DOT	300ft	100ft	300ft	100 ft	Depends on Conditions	
Nova Scotia DOT	60 m		60 m		60 m	
Ohio DOT	660ft Diam Int	1000ft Dirc Int	660ft Diam Int	1000ft Dirc Int	660ft Diam Int	1000ft Dirc Int
Oregon DOT	1320ft	750ft	1320ft		1320ft min statewide intersections	1/2 mile on regional highways
South Carolina DOT	300 ft	100 ft	300ft	100ft	300ft	100 ft
South Dakota DOT	660ft		660ft		1320ft	
Utah DOT	100m	50m	100m	50m	100m	50m
Vermont	500 ft	250 ft			500 ft	250 ft
Virginia DOT	300ft	100ft	300ft	100ft	300ft	100 ft
Washington State DOT	130ft minimum , 300 ft prefered		130ft minimum, 300ft prefered		1/2 mile	
West Virginia DOT	300ft	100ft	300 ft	100ft	300ft	100ft
Wisconsin	1000 ft arterials , collectors/500ft		1000 ft arterials , collectors/500ft			
Wyoming DOT	300ft	150ft	300ft	150ft	300 ft	150 ft

Crash Models

Several research efforts have been conducted to determine the relationships between crash rate and traffic and geometric characteristics. Independent variables taken into account in safety models include: traffic volume (AADT, peak volume, percentage of trucks, etc.), section length, number of lanes, width, type of median, curvature, existence of left or right turn, etc.

Williams et al. (2004) studied safety data for a one-mile freeway section before off-ramps (segment to most likely have safety problems due to insufficient access-controlled right of way) for 11 sites and found that the potential number of crashes could be reduced when signalized access spacing is increased Figure 8.

The researchers also suggested that the public may benefit greatly by increasing the minimum access to a minimum of 600 ft.

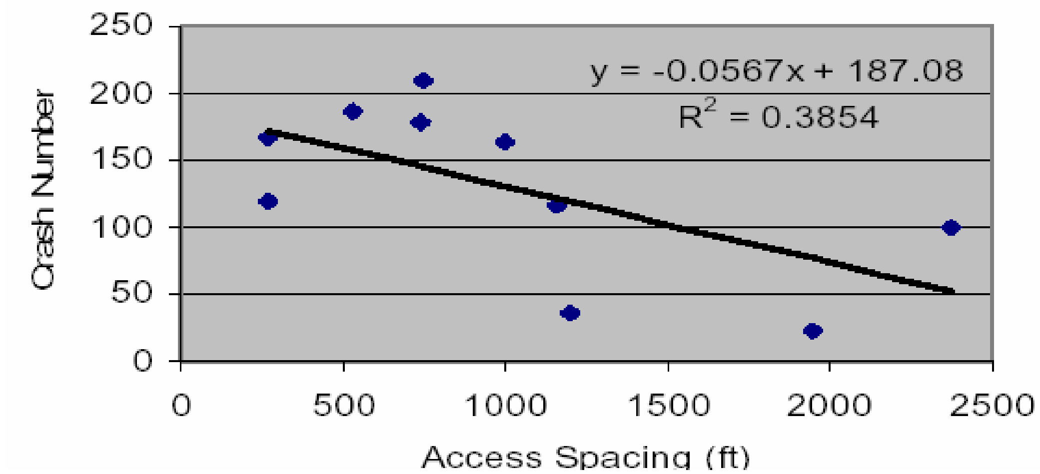


Figure 8. Effects of Access Spacing on Number of Crashes.

Source: Williams, K., H. Zhou, L. Haggen, and W. Farah. Benefit and Cost Analysis of Strategic Acquisition of Limited Access Right of Way near Interchanges. *Presented at the 6th National Access Management Conference*, Kansas City, Missouri, 2004.

The current state of the art for developing crash prediction models (CPMs) is to adopt general linear models (GLMs) considering either a Poisson or a negative binomial error structure (Lord, 2004; Lord, 2005; Sawalha, 2006).

Any model selected must satisfy two basic conditions (Cameron et al., 1998; Miau et al., 1993): it must yield logical results and a known link function must exist that can linearize this form for the purpose of coefficient estimation. To satisfy the first condition, the model must not lead to the prediction of a negative number of accidents and must ensure a prediction of zero accident frequency for zero values of the exposure variables (section length and AADT).

However, when selecting explanatory variables, the model must be developed following the principle of parsimony; that is, to try to explain as much of the variability of the data using the least number of explanatory variables. This is a characteristic that is required especially if the model will be used for the safety study of new locations not included in its development (Sawalha, 2003).

Recently, researchers have also proposed the use of zero-inflated negative binomial regression models in order to address the high propensity of zero crashes within typical crash data (Shankar et al., 1997; Shankar et al., 2003).

The use of linear regression models (LRMs) is not utilized because crash data typically do not satisfy the assumptions of such models, namely: normal error structure and constant error variance. In general, linear regression models are usually not appropriate for count response data unless the mean response is relatively high

An earlier publication (Lord et al., 2004) indicated that "there has been considerable research conducted over the last 20 years focused on predicting motor vehicle crashes on transportation facilities. The range of statistical models commonly applied includes binomial, Poisson, Poisson-gamma (or Negative Binomial), Zero-Inflated Poisson and Negative Binomial Models (ZIP and ZINB), and Multinomial probability models. Given the range of possible modeling approaches and the host of assumptions with each modeling approach, making an intelligent choice for modeling motor vehicle crash data is difficult at best." The authors further indicate that "in recent years, some researchers have applied 'zero-inflated' or 'zero altered' probability models, which assume that a dual-state process is responsible for generating the crash data." The authors indicated that "these models have been applied to capture the 'excess' zeroes that commonly arise in crash data—and generally have provided improved fit to data compared to Poisson and negative binomial (NB) regression models."

Lord et al. (2004) conducted a simulation experiment to demonstrate how crash data may give rise to "excess" zeroes. They demonstrated that under certain (fairly common) circumstances excess zeroes are observed—and that these circumstances arise from low exposure and/or inappropriate selection of time/space scales and not an underlying dual state process. They concluded that a careful selection of the time/space scales for analysis, including an improved set of explanatory variables and/or unobserved heterogeneity effects in count regression models, or applying small area statistical methods (observations with low exposure) represent the most defensible modeling approaches for datasets with a preponderance of zeros. We partially agree with these conclusions; however, modelers may not have much choice in their time/space scale selection given the limitation of traffic and crash data.

Data Collection

This section describes in detail the methodology used to capture geometric, traffic, and crash data for the study (Stage 1, Figure 1). The objective of this effort was to compile all data relevant for the safety analysis. The original research plan called for the data collection of 12 interchanges; however, due to the fact that accidents are rare occurrences, the research team expanded the project scope to collect data for 186 locations. These locations were selected at random; and thus it can be argued that these locations are representative of the state's interchanges and access geometrics, which increases the probability of producing statistically reliable results.

For the data collection process, the VDOT Geographic Information System (GIS) Integrator platform for the management of geospatial information was used (Figure 9). This platform stores roadway network and aerial photography information. Aerial photos were used to obtain the geometric attributes that define characteristics of the off-ramp and the cross-section. In addition, the database provides a visual record of each highway and a windshield view of what drivers see when traveling the road. These video logs were used to identify the elements of the road and ramp that were not visible from the aerial photo.

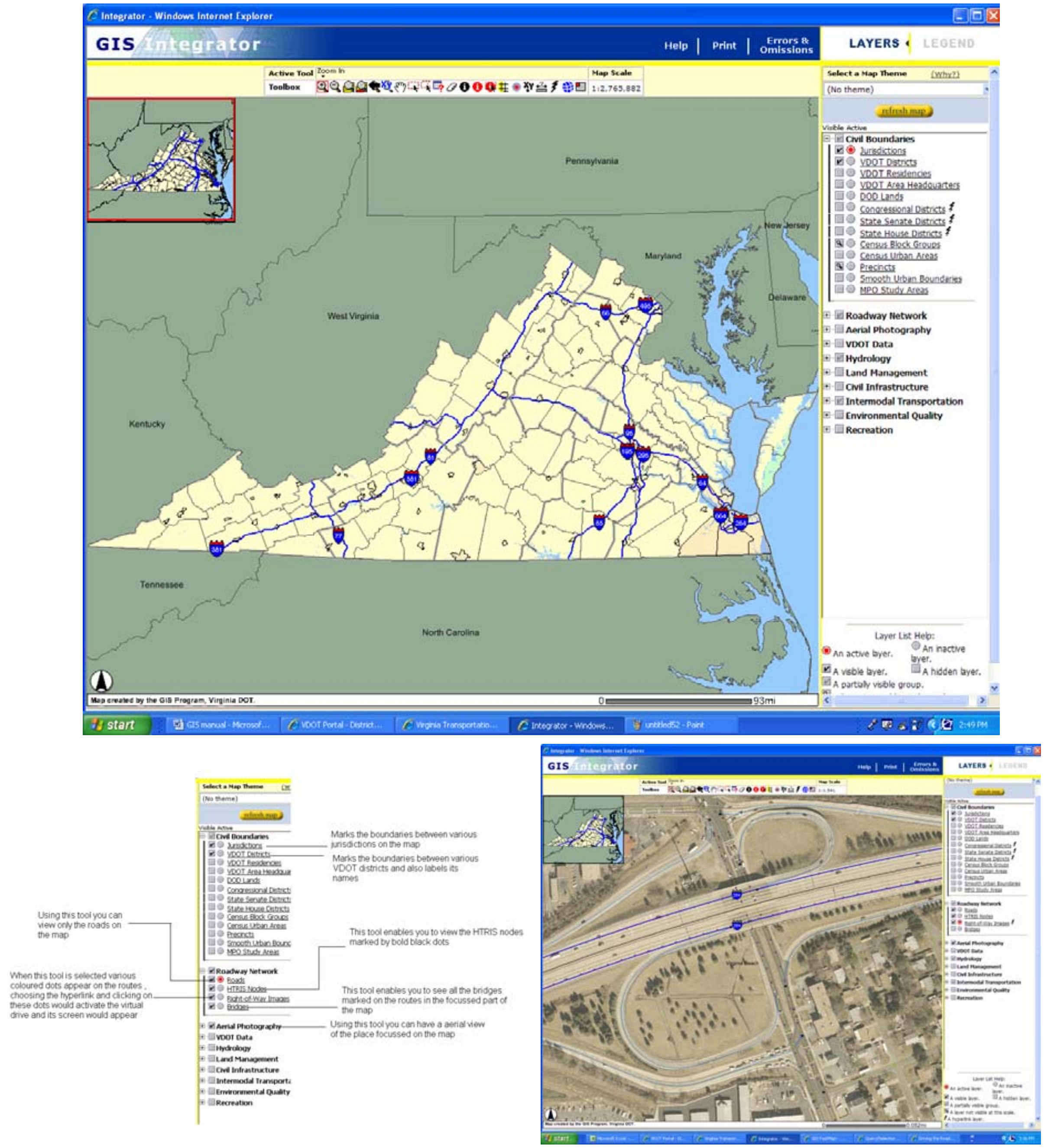


Figure 9. GIS Integrator

Geometric and Traffic Data

The following information was collected for each location: interstate description, exit and ramp ID, reference node numbers (intersection ramp and crossroad, and crossroad and first intersection), presence of acceleration lane, median and left turn lane, type of first access (right, left, intersection), distances from the off-ramp to the first access (L_1), first intersection distance from the off-ramp (L_2), first median opening (M) and the on-ramp, distance from the last access before the on-ramp to the on-ramp (Z), type of traffic control at the first intersection, etc. The characteristics of the sections are shown in Figure 10 and Figure 11 and described in detail later in this report.

Figure 10. Aerial Photos and Video Logs

The average distance to the first access was 169 m (550 ft) and the average distance to the first intersection was 298 m (978 ft); the distribution is shown in Figure 12.

The traffic data were obtained from the road inventory database. The data represent the Average Annual Daily Traffic (AADT) of the section with a few exceptions where the traffic data represent a 24-h count. The average AADT was 19,000 veh/day with 70% of the sections having an AADT of less than 25,000 veh/day, as illustrated in Figure 12.

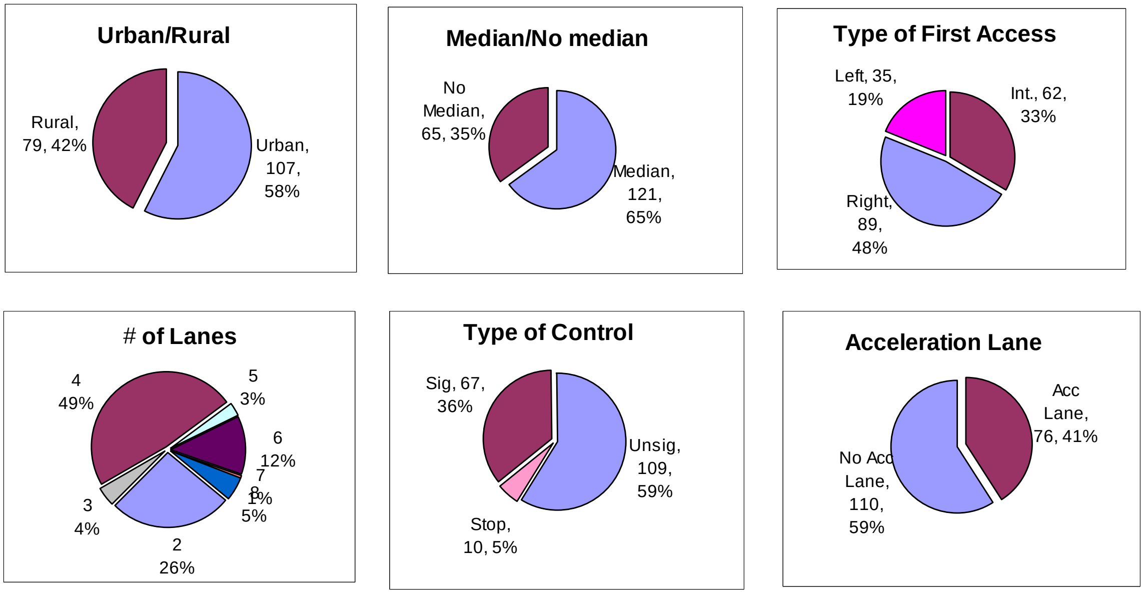


Figure 11. Geometric Characteristics of the Data

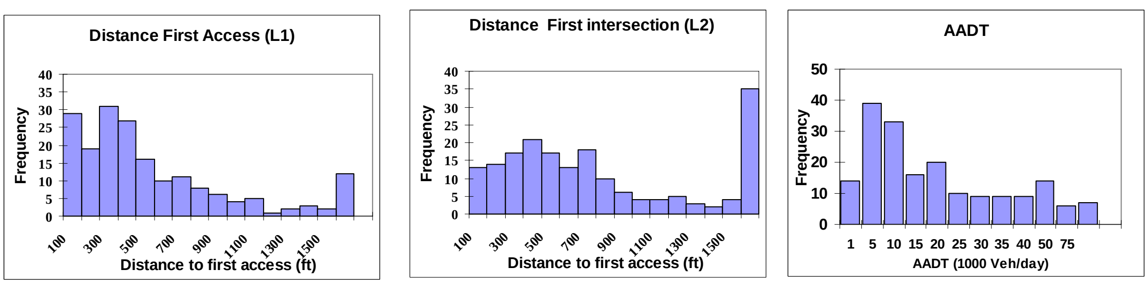


Figure 12. Distance to the First Access Distribution

Crash Data

VDOT locates each crash occurring on a VDOT-maintained roadway through an interface with the Highway Traffic Records Information System (HTRIS). The HTRIS contains point locations and related data for all reportable motor vehicle accidents in Virginia from January 1997 through December 2005. A reportable accident must have a fatality, an injury, or property damage in excess of $1000. The approximate mile points assigned to each accident are derived from the forms submitted by the police/public safety personnel. The system generates an equivalent mile point for the crash. Through an extract program, certain data elements are brought over from the CAP System (Common Alerting Protocol) to populate the Accident Records Sub-system of HTRIS. The upstream node information from the GIS Integrator and road inventory of the crash database was used to search the accident file to collect accident data from 2001 through 2005 for all of the sections. The accident record contains location, date, type of accident, driver, and vehicle information. The total number of crashes for all of the sections was 2,277 over five years (Figure 13). The average number of crashes per site was 12.24 per five-year exposure, with a minimum of zero and a maximum of 168 crashes. The crashes were segregated as total, in the crossroad (access) and before the first access (BFA). In addition, the crashes were further segregated into (1) injury and fatal crashes and (2) property damage only crashes.

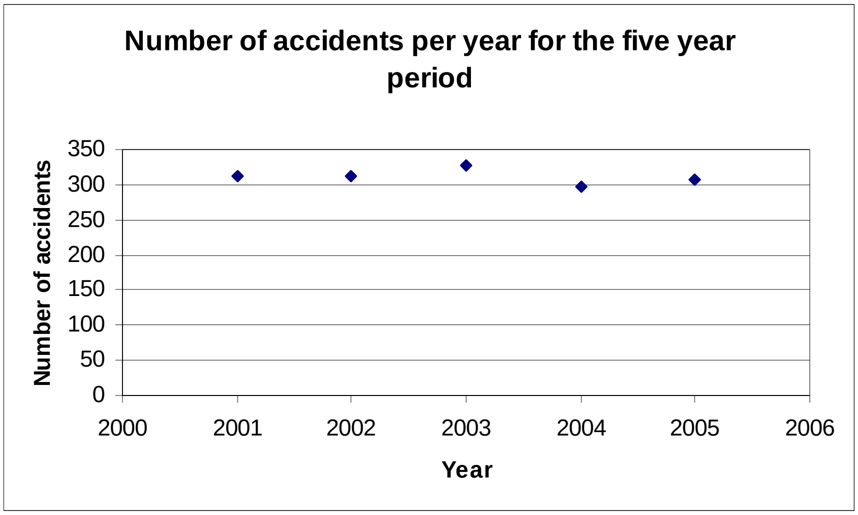


Figure 13. Five Years' Crash Distribution

The traffic data were obtained from the road inventory file. The data represent the AADT of the section with a few exceptions where the traffic data represent a 24-hour count.

Figure 14 shows the variation in the total number of crashes, the number of crashes on the access road, and the number of crashes before the first access road as a function of the AADT. The figure demonstrates an increase in the number of crashes as the AADT increases; however, the relationship does not appear to be linear. According to previous studies, crashes increase with traffic in a non-linear fashion. Qin et al. (2002) concluded that the assumption that AADT and segment length have the same exponents in the exposure function is doubtful, and that the number of crashes and AADT is non-linear when different crash types are considered. These factors were considered when the model was developed.

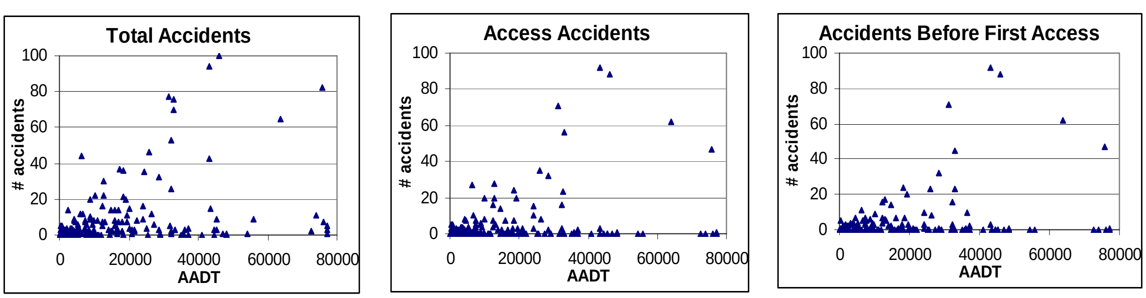


Figure 14. Number of Accidents versus AADT

After fusing the crash, traffic, and geometric data it was possible to plot the data, as illustrated in Figure 15. Specifically, the figure demonstrates a general increase in the number of crashes as the facility AADT increases. The figure also illustrates a high cluster of data at the short access road distances with minimum observations for access roads in excess of 400 m. Similarly, AADTs in excess of 8,000 veh/day are a rare occurrence. The figure illustrates a number of sections with high AADTs and short access roads with a small number of crashes. Conversely, rare observations with high crashes are also observed for low AADTs and long access roads.

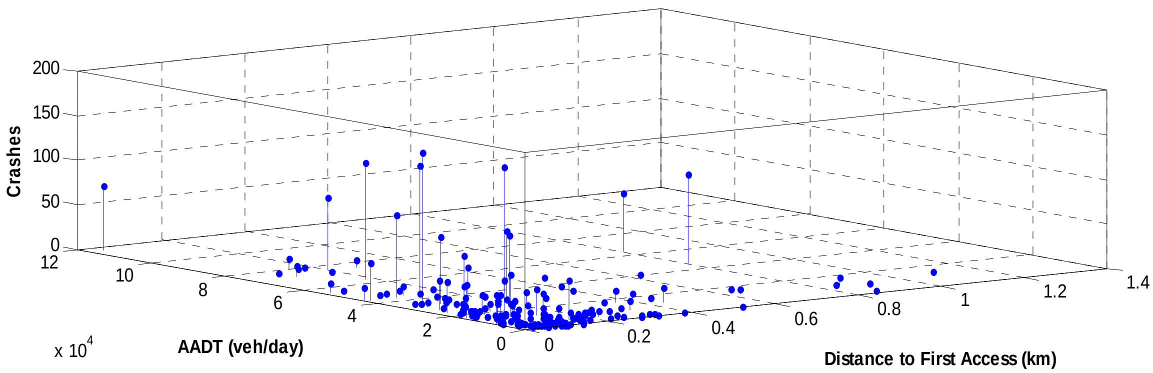


Figure 15. Number of Crashes as a Function of AADT and Distance to the First Access

Figure 16 clearly shows an inverse relationship between the number of crashes and the distance from the off-ramp to the first access road (L1) for the different types of crashes. Fatal crashes were very rare occurrences and injury and property crashes represent 65% and 35% of the total crash database, respectively. These percentage breakdowns are consistent with the statewide breakdown.

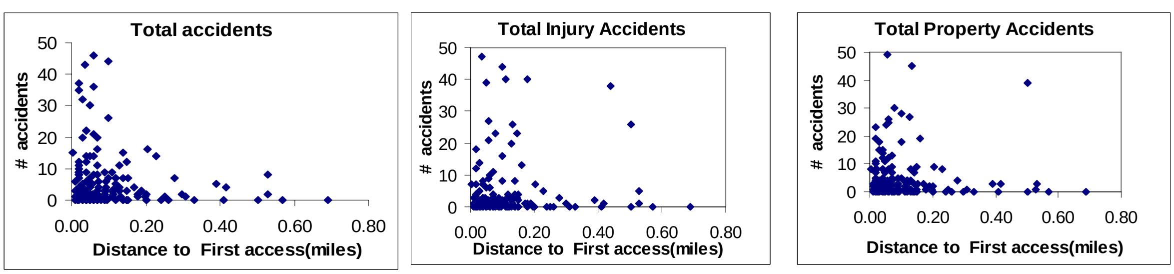


Figure 16. Number of Crashes for different type of Accidents as a Function of the Distance to the First Access

Exploratory Data Analysis

In this section we present a qualitative data analysis conducted to identify the impact of geometric and traffic factors on the crash rate as a function of the distance from the off-ramp to the first access road. In this analysis the distance to the first access road is considered from the gore of the off-ramp to the centerline of the access road. Additional analysis results are presented in Appendix (B) considering the distance to the first access road for the end of the acceleration lane taper.

The crash rate is computed as

$$CR = \frac{C}{(AADT \times 365 \times 5) \times 10^{-6} \times L_2} \qquad [4]$$

where CR is the crash rate in crashes per million-veh-mi or million veh-km; C is the number of crashes along a section, and L_2 is the length of the section which is the distance between the freeway off-ramp and the first intersection (mi or km).

Crash rates were computed for each of the different categories. For each category the 300, 660, 750, 990 and 1320 ft, (90, 200, 230 and 400 m) lines are also plotted to reflect the current Virginia standards and more conservative ones. In addition, for each category, the average crash rate and the number of sections with distances to the first access road of less than 300 ft (90m) d1, and between 300 ft (90 m) and 750 ft (230 m) d2, were computed to show the impact of extending the access distance (Table 6 and Table 7).

Table 6. Average Crash Frequency, Rate, Distances and AADT for Different Categories

Category	Sub cat.	Number Sections	%	Crash F[1]	CR[2]	CR d1[3]	Number Sections	CR d2[4]	Number Sections	L1(mile)[5]	L2(mile)[6]	AADT
Total	Total	186	100	12,24	6,25	10,61	79	3,85	68	0,11	0,19	19456
Area	Urban	107	58	17,61	5,05	8,61	45	2,90	35	0,11	0,19	28299
	Rural	79	42	4,97	7,89	13,26	34	4,85	33	0,11	0,18	7479
Median	Yes	126	65	14,85	4,29	7,75	40	3,05	49	0,12	0,18	25882
	No	60	35	7,38	9,91	13,54	39	5,90	19	0,08	0,20	5962
Type of Control First access	Signalized	67	36	17,61	4,06	7,13	24	3,03	25	0,13	0,20	30063
	Unsignalized	119	64	9,22	7,49	12,36	50	4,55	40	0,09	0,18	13485
LTL	Yes	78	42	18,45	4,02	7,06	23	3,43	32	0,13	0,20	28581
	No	108	58	7,76	7,87	12,07	56	4,21	36	0,09	0,18	12866
Acceleration Lane	Yes	76	41	18,54	5,14	10,67	26	3,27	28	0,13	0,20	31399
	No	110	59	7,89	7,03	10,58	53	4,25	40	0,09	0,18	11205
Number of lanes	2	57	31	4,98	9,44	12,64	36	5,26	15	0,08	0,22	5204
	4	95	51	13,89	5,65	10,51	33	3,94	40	0,12	0,16	17744
	More than 6	34	18	19,79	2,6	3,64	10	2,39	17	0,11	0,20	48133
Type of First Access	Intersection	62	33	17,69	4,79	13,02	11	4,08	30	0,16	0,16	29861
	Right	89	48	9,80	6,66	9,25	50	4,23	27	0,07	0,19	16065
	Left	35	19	8,80	8,19	13,43	18	2,58	11	0,09	0,23	9648

(1) Average Crash Frequency for 5 years exposure
(2) Average Annual Crash Rate Crashes per million-veh-miles
(3) d1 Distance from the off ramp to the first access between 0-300 feet
CR d1 Average Annual Crash Rate for sections with access distances for sections with distance from the off-ramp to the first access between 300-750 ft
(4) d2 distance from the off-ramp to the first access between 300-750 ft
CR d2 Average Annual Crash Rate for sections with access distances from the off-ramp to the first access between 300-750 ft
(5) L1 Distance from the off-ramp to first access (miles)
(6) L2 Distance from the off-ramp to first intersection (miles)

Table 7. Average Crash Rate for Different Categories

Category	Sub category	Total	Access	BFA	Total [1]	Access [1]	BFA [1]	Total [2]	Access [2]	BFA [2]
Area	Urban	7.89	4.35	5.52	8.61	3.05	3.77	2.47	1.55	1.61
	Rural	5.05	2.18	2.51	13.26	6.74	8.83	3.83	2.55	5.75
Median	Yes	4.29	2.21	2.43	7.75	3.00	4.71	2.83	1.99	4.86
	No	9.99	4.72	6.27	13.54	6.27	4.88	4.44	2.40	4.73
Control 1st access	Sig	4.06	1.43	1.34	7.13	1.41	1.11	2.35	1.44	1.47
	Unsig	7.49	3.96	4.31	12.36	5.83	6.29	3.65	2.37	2.62
LTL	Yes	4.02	1.81	1.93	7.06	1.80	2.20	3.35	2.14	2.61
	No	7.87	4.04	5.14	12.06	5.81	7.48	2.75	1.81	1.82
Acceleration Lane	Yes	5.14	1.94	2.06	10.67	2.36	2.79	2.47	1.68	1.62
	No	7.03	3.72	4.70	10.58	5.64	7.32	3.38	2.14	2.55
Number of lanes	2	9.44	5.07	7.14	12.63	7.14	9.85	3.95	1.53	2.49
	4	5.65	8.51	2.45	10.51	2.68	2.43	3.88	3.11	3.15
	>6	2.60	1.48	1.94	3.64	2.09	3.49	2.17	1.22	1.29
Type of First Access	Int	4.79	2.14	2.13	12.94	4.96	4.96	3.00	1.53	1.52
	Right	6.66	3.33	3.80	9.25	4.03	4.66	3.31	2.44	2.70
	Left	8.19	4.14	6.59	13.43	6.03	9.98	2.62	2.13	3.01

(1) Average Annual Crash Rate for sections with distance from the off-ramp to the first access between 0-300 ft.
(2) CR d2 Average Annual Crash Rate for sections with distance from the off-ramp to the first access more than 300 ft.

Rural vs. Urban

Rural sections comprise 42% of the sites while urban sections comprise 58% of the sites. The rural sections have a higher average crash rate when compared to urban sections with values of 7.89 and 5.05 crashes per million vehicle kilometers of travel, respectively (Figure 17). The higher crash rates for the rural category are basically due to low exposure because the average numbers of crashes are 5 and 17 for rural and urban roads, respectively. However, these differences are not statistically significant (p value of 0.12). For sections, where the first access is located less than 300 feet from the off-ramp, the crash rate is 13.26 for rural roads and 4.85 for urban roads. When the distances to the first access are between 300 and 750 feet there is a significant reduction (>50%) in the crash rates compared to the distances to the first access are less than 300 feet.

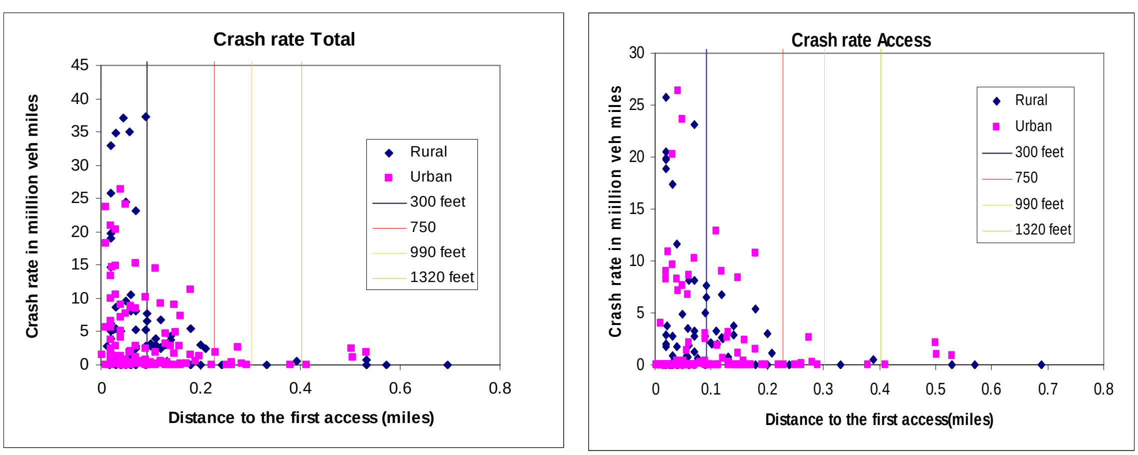


Figure 17. Crash Rate for the Urban/Rural Category

Median

Several studies have shown the positive impact of medians on the safety of the roadways (Gluck et al., 1999; Sawalha et al., 2000). The highest accident rate is experienced when there is no median, basically because the median separates both directions of travel and reduces left turns and other movements across the median. The crash rate without medians is 9.91 compared with 4.20 for sections with medians (Figure 18). The differences of the means are statistically significant (p value of 0.0066). For sections with distances to the first access shorter than 300 feet, the crash rate is 13.54 for sections with no median, and 7.75 with a median. For access distances between 300 and 750 feet, the numbers are reduced to 5.9 and 3.05, respectively.

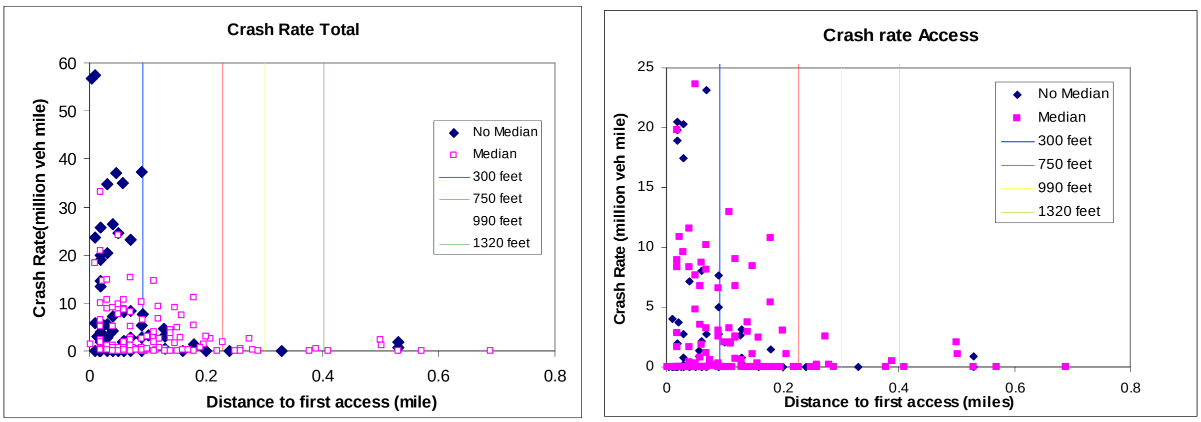


Figure 18. Crash Rate as a Function of the Presence of a Median

Type of First Access

The aerial photography and the video logs were not reliable enough to identify in all cases if the first right or first left access were private or commercial in nature. The nature of a commercial business can result in higher traffic demand than a normal crossroad. For this reason some sections were reclassified. For example, an intersection classified as an unsignalized left T intersection was considered in this category as a left first access. The first classification took precedence so, if there was a right entrance before the unsignalized left T intersection with the crossroad, it continued to be classified as a right entrance. The average crash rate for all of the accidents was found to be lower when the first access was an intersection (4.79) and higher for right and left access roads (4.14 and 5.09, respectively) (Figure 19). The average crash rates, for those sections with distances to the first access between 300 and 750 feet, experienced a significant reduction compared with the ones with access distances less than 300 ft with more than 50% reduction for all three cases.

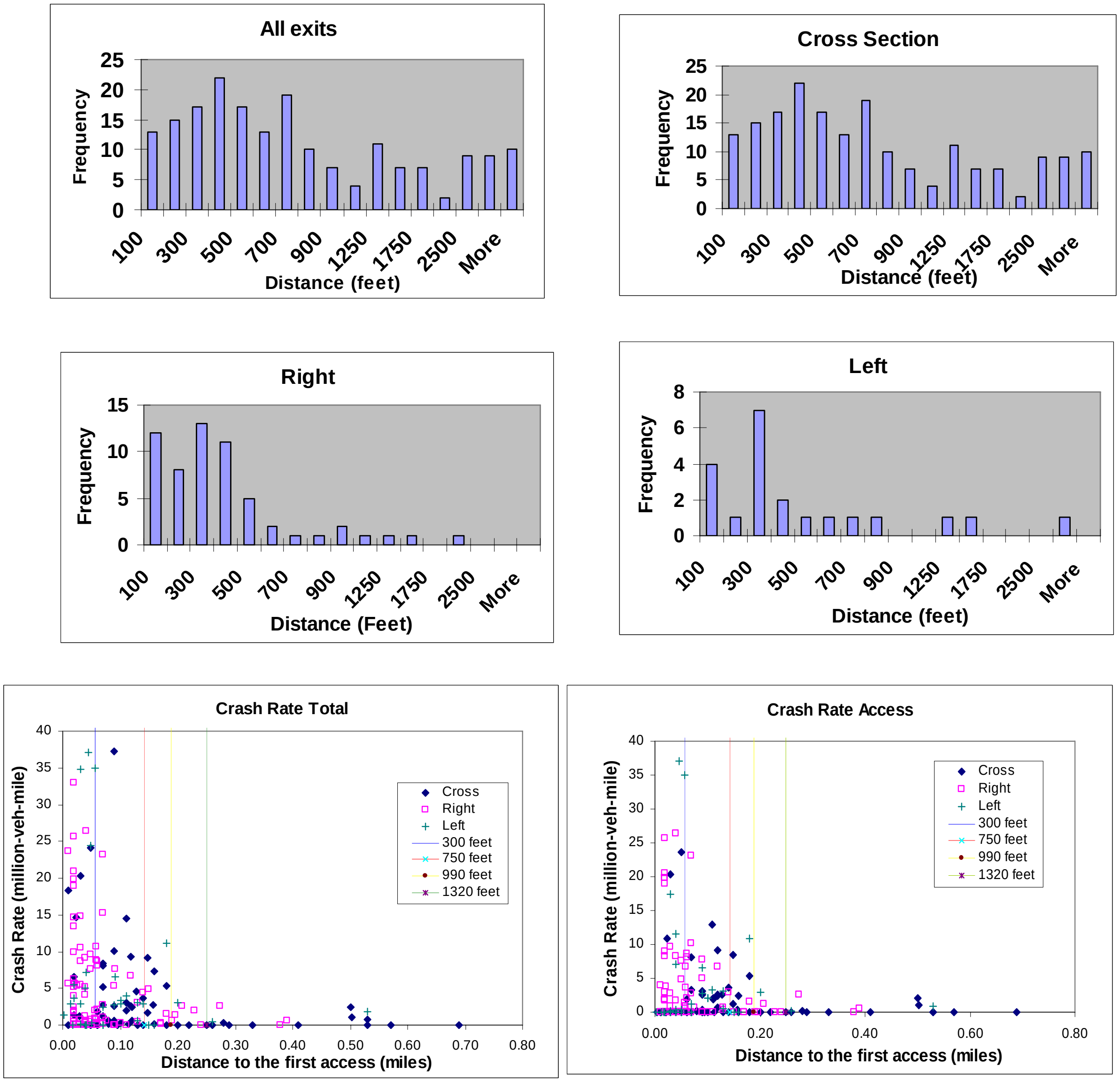


Figure 19. Distribution of Distances and Crash Rate for the Type of Access Category

Number of Lanes

The crash rate distribution based on the number of lanes is shown in Figure 20. Four-lane sections represent the majority of the sections of the study (51%) followed by two lanes (27%). Noland et al. (2004) found that increasing the number of lanes on a given road was associated with more accidents. The average number of crashes in this study increases with the number of lanes, from 4.98 for two lanes to 14.04 for four lanes and 23.43 for six lanes. The crash rate for the system decreases from 5.7 to 3.54, 1.81, and 1.2, respectively. The average rates for sections with distances to the first access of less than 300 feet are 12.64 for two lanes, 10.51 for four lanes, and 3.64 for sections with six or more lanes. For the sections where this distance is increased between 300 and 750 feet, there is a decrease in the crash rate to 5.26, 3.94, and 2.39, respectively.

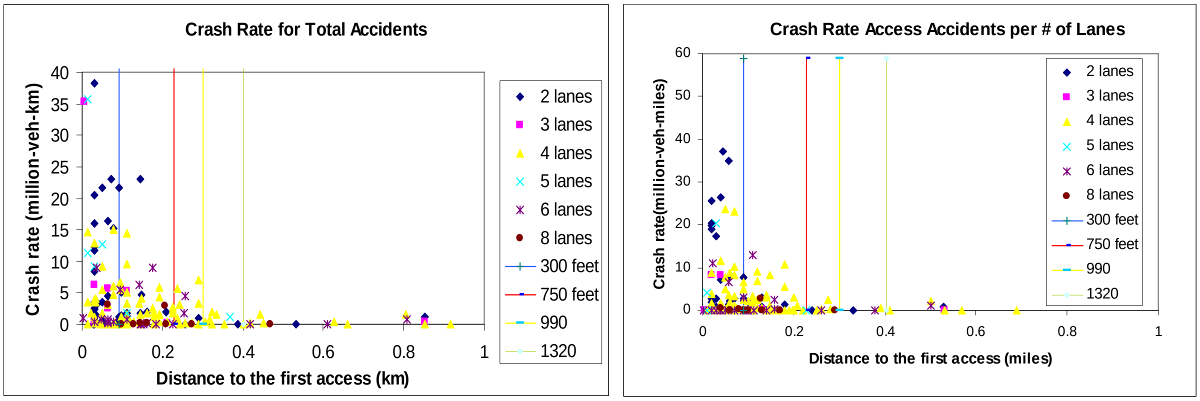


Figure 20. Crash Rate for the Number of Lanes Category

Left Storage Lane

The provision of left turn storage lanes at the first intersection has been found in the literature to reduce accident rates in the range of 20 to 65% (NYDOT, 1990; FHWA, 1992; Hummer et al., 1994). Similar reductions were found in this study, with the average crash rate of 7.87 for sections without a left turn lane compared with 4.02 for sections with a left turn lane. The differences are statistically significant (p value of 0.01). Sections without left turn lanes with distance to the first access shorter than 300 feet experience an average crash rate of 12.07 (Figure 21). This crash rate is reduced by 65% for sections with distance to the first access between 300 and 750 feet. In the case of sections with a left turn lane, the reduction is 50%.

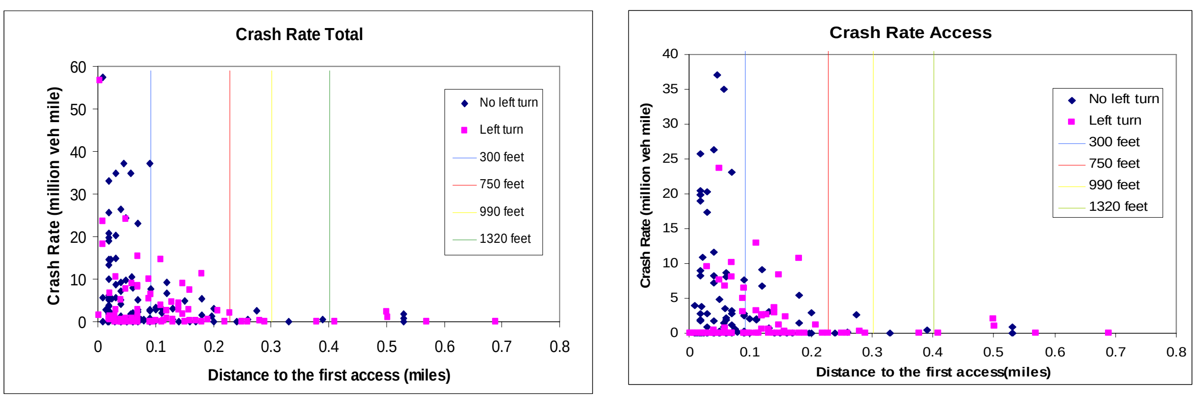


Figure 21. Crash Rate as a Function of the Presence of a Left Turn Storage Lane

Type of Control at First Intersection

Unsignalized intersections are the most common ones after the off-ramps (59%) followed by signalized intersections (36%) and stop signs (5%). The average crash rate was 7.49 for unsignalized sections and 4.06 for signalized sections. The crash rate for sections with distances to the fist access less than 300 feet was 12.07, with 44% of the unsignalized sections in this category. For sections with access distances between 300 ft and 750 ft, the rate was reduced by 65%. A similar situation is experienced for signalized sections with a drop in the crash rate in the range of 62% (Figure 22).

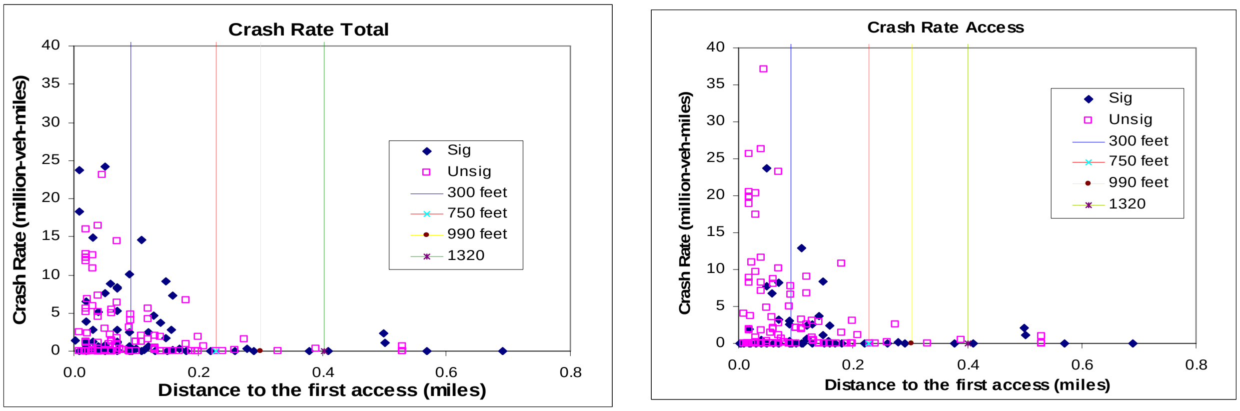


Figure 22. Crash Rate for the Signalized/Unsignalized Category

Acceleration Lane

Seventy-six of the locations had acceleration lanes. Sections with no acceleration lane have a higher crash rate (7.03) compared with the sections that have an acceleration lane (5.14). The rate for distances to the first access shorter than 300 feet are almost the same in both cases; however, for access distance between 300 and 750 feet, sections with acceleration lanes experienced bigger reductions (69%) than the ones that do not have an acceleration lane (Figure 23).

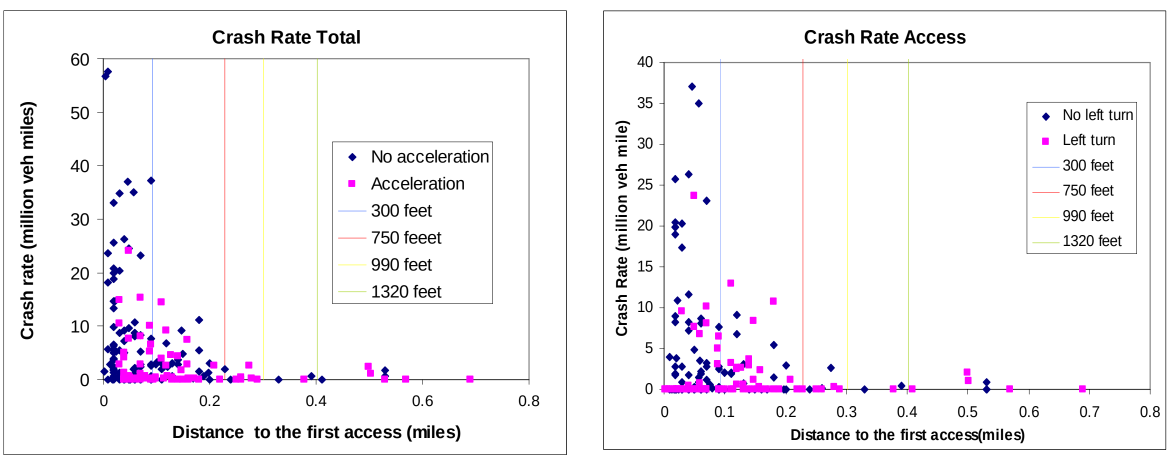


Figure 23. Crash Rate As a Function of the Presence of Acceleration Lane

Crash Models

Several research efforts have been conducted to determine the relationships between crash rate and traffic and geometric characteristics. As mentioned in the literature review, LRMs are not used because crash data typically do not satisfy the assumptions of such models, namely, normal error structure and constant error variance. Linear regression models need to fulfill the conditions of normality and homoscedasticity of the data (constant variance) and, as illustrated in Figure 24, the count (crash) data did not pass the normality test.

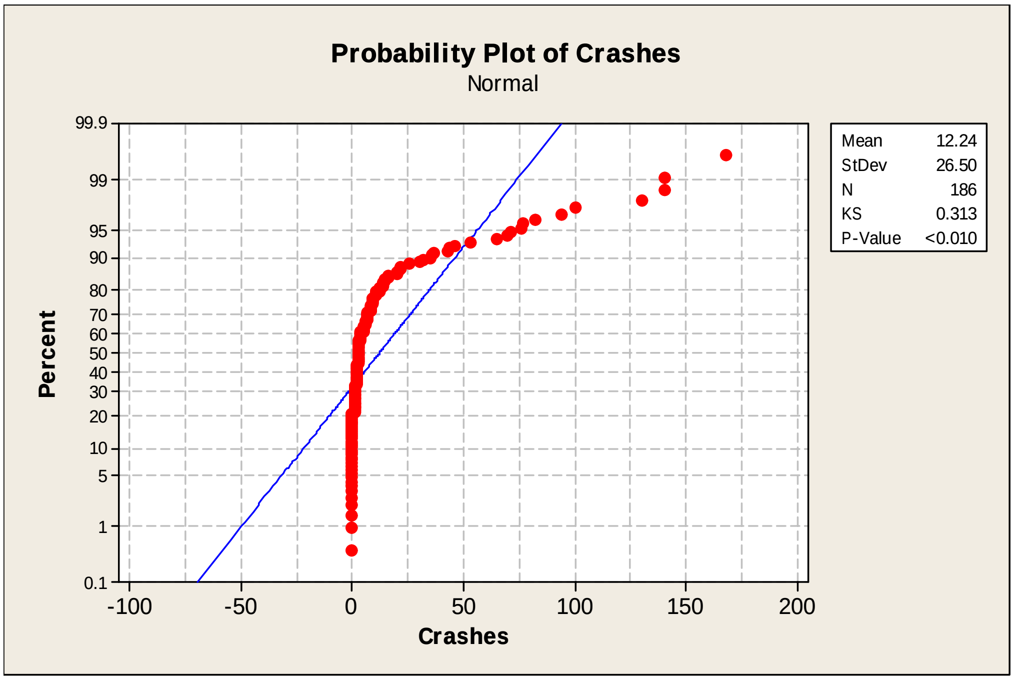


Figure 24. Normality Test

In this report we present two approaches that were tested for developing crash prediction models. The first approach is the common approach that is reported in the literature, which is based on the use of Poisson or negative binomial regression models. An alternative approach that is developed in this report is the use of LRMs.

Model Structure

Prior to describing the various models, the model structure is discussed. Specifically, the study considers a crash rate that is formulated as

$$CR = \frac{C}{L_2 V^p} \times \frac{10^6}{(365 \times 5)^p} = \exp(b_0 + b_1 L_1), \qquad [5]$$

where CR is the crash rate (million vehicle crashes per vehicle kilometer of exposure over a 5-year period), C is the total number of crashes over the study section of length L_2 in the 5-year analysis period (crashes), L_2 is the length of the section which is the distance between the freeway off-ramp and the first intersection (km), L_1 is the distance between the freeway off-ramp and the first access road (may equal L_2 if the first access road is an intersection) (km), V is the section AADT (veh/day), and B_0 and B_1 are the model constants.

The model of Equation 6 can then be manipulated to produce a log-transformed linear model of the form

$$C = \frac{(365 \times 5)^p}{10^6} \times \exp\left(b_0 + b_1 L_1 + \ln(L_2) + p\ln(V)\right). \qquad [6]$$

The advantage of this model is that it is linear in structure after applying a logarithmic transformation; it ensures that the crashes are positive (greater than or equal to zero); and) it produces zero crashes when the exposure is set to zero (i.e., when L_2 or V is zero).

Poisson or Negative Binomial Model Approach

Because crash frequencies take on non-negative integer values, count data modeling techniques such as Poisson and NB regression are obvious choices (Milton et al., 1998). The probability mass function of a Poisson (λ) random variable is given by:

$$P(n_i) = \frac{\lambda^{n_i}}{n_i!} e^{-\lambda} \quad \text{where } n_i = 0, 1, 2, \ldots. \qquad [7]$$

where $P(n_i)$ is the probability of n accidents occurring on a highway section i and λ is the expected accident frequency for highway section i. In applying the Poisson model, the expected accident frequency is assumed to be a function of explanatory variables such as

$$\lambda_i = e^{BX_i} \qquad [8]$$

where X_i is a vector of explanatory variables that can include geometric and traffic parameters for section i that determine the accident frequency and B is a vector of model parameters that are estimated using maximum likelihood methods.

One feature of the Poisson random variable (y) is the identity of the mean and variance. Two Poisson regression models were developed, one with and one without the zero crash data. The Poisson models were fit to the data to derive

$$\frac{C}{L_2} = \frac{1825^p}{1000000} \times \exp\left(b_0 + b_1 L_1 + p\ln(V)\right) \qquad [9]$$

where C is the total number of crashes over the study section of length L_2 in the 5-year analysis period (crashes); L_2 is the length of the section which is the distance between the freeway off-ramp and the first intersection (km); L_1 is the distance between the freeway off-ramp and the first access road; V is the section AADT (veh/day); and B_0 and B_1 are the model constants.

The standard Poisson regression model assumes equi-dispersion (variance equals mean). Cameron and Trivedi (1990) suggest a quick check for over-dispersion. If the variance of the count data exceeds twice the unconditional mean, the data is considered over-dispersed. Although this is a quick diagnostic, more rigorous approaches to test for over-dispersion must be conducted. To estimate the dispersion parameter, the deviance or scaled Pearson's chi-square statistics divided by its degrees of freedom can be used. If the estimate of the dispersion parameter is substantially larger than 1, over-dispersion is present in the data.

Assessing the goodness of fit for the model, one can observe that the Poisson regression model suffers from over-dispersion as indicated by the value of the deviance divided by the degrees of freedom which is much greater than 1. For this reason two different approaches were used: the negative binomial and the modified Poisson regression using an over-dispersion parameter. The negative binomial regression model with the mean dispersion function can be viewed as an expansion of the Poisson model because it adds a random disturbance to the exponential mean function of the Poisson model. Therefore, it is a good alternative model for count data that exhibit excess variation compared to the Poisson model.

Variable Impacts

To study the impact of each variable on the final model a dummy variable was defined as

$$D = \begin{cases} 1, & \text{median} \\ 0, & \text{no median} \end{cases}$$

to derive a full model as $CR = \exp(\beta_0 + \beta_1 L_1 + \beta_2 D + \beta_3 (D \times L_1))$ and a reduced model of $CR = \exp(\beta_0 + \beta_1 L_1)$.

The model can be cast as

$$D = \begin{cases} 1, & CR = \exp((\beta_0 + \beta_2) + (\beta_1 + \beta_3) L_1) \\ 0, & CR = \exp(\beta_0 + \beta_1 L_1) \end{cases} \qquad [10]$$

A test for the coincidence of the two regression models was performed. This test evaluates the equality of the intercepts and slopes simultaneously. The null hypothesis tested is that there is no difference between these models. This test is evaluated using the following statistical test, which follows the F-distribution as

$$F = \frac{Dev_{Reduced} - Dev_{Full}}{r \times \hat{\phi}} \sim F_{(df_1, df_2, \alpha)} \qquad [11]$$

where $Dev_{Reduced}$ is the calculated deviance for the reduced model; Dev_{Full} is the calculated deviance for the full model; d_{f1} is the number of parameters being tested; d_{f2} is the degrees of freedom for the full model (n-p); and $\hat{\phi}$ represents the estimated dispersion parameter.

The deviance of a GLM is derived from the likelihood principle. It is proportional to twice the difference between the maximized log-likelihood evaluated at the estimated means $\hat{\mu}_i$ and the largest achievable log-likelihood obtained by setting $\hat{\mu}_i = y_i$.

The deviance for the Poisson distribution is given by

$$D = 2\left[\sum_{i=1}^{n} y_i \ln\left(\frac{y_i}{\hat{\mu}_i}\right) - (y_i - \hat{\mu}_i)\right] \qquad [12]$$

As the model fit ($\hat{\mu}_i$) gets closer to the actual value of the response variable (y_i), the deviance gets smaller. On the contrary, a model that poorly fits the data results in a larger value of the deviance. From Equation 12 one can see that the term $(y_i - \hat{\mu}_i)$ represents the residuals. Therefore, the deviance is used instead of the sum squares. For example, in the Gaussian data with identity link, the deviance is an error sum of squares.

Linear Regression Modeling Approach

A linear regression approach for the development of a model was considered as an alternative option. If we consider the number of crashes per unit distance as our dependent variable the model of Equation 6 can be cast as

$$\frac{C}{L_2} = C' = \frac{(365 \times 5)^p}{10^6} \cdot \exp\left(b_0 + b_1 L_1 + p\ln(V)\right). \qquad [13]$$

Equation 13 is an exponential model with two independent variables: V and L_1. It should be noted that an analysis of crashes per unit distance ensures that the data are normalized across the different section lengths. The development of an LRM using the least squares approach requires that the data follow a normal distribution. A statistical analysis of the data revealed that there was insufficient evidence to conclude that the data were normal. Furthermore, the dispersion parameter, which measures the amount of variation in the data, was significantly greater than 1.0 indicating that a negative binomial model would be appropriate for the data.

Normalization of the Data

Here we present an approach for normalizing the data in order to apply a least-squared LRM to the data. The approach involves sorting the data based on one of the independent variables and then aggregating the data using a variable bin size to ensure that the second independent variable remains constant across the various bins. Data transformations can then be applied to the data to ensure normality and homoscedasticity (equal variance). Once the parameters of the first independent variable are computed, the data are sorted on the second independent variable. The data are then aggregated in order to ensure normality and homoscedasticity and then linear models are fit to the data to compute the variable coefficient. The approach is demonstrated using the access road crash data in the following sub-sections.

Selecting Exposure Measures

The typical exposure measure for crashes is million vehicle-miles. However, researchers have argued that the exponent of the volume variable (V) in the exposure measure is not necessarily equal to 1.0 (Quin et al., 2004; Ivan, 2004). Consequently, the first step in the analysis was to compute the exponent of V (denoted as p).

In estimating crash rates it is important that the measure of exposure ensures that the data are normalized. In doing so a multiplicative crash adjustment factor (F_i) for each bin i was computed as

$$F_i = \frac{\min_i \left[\max_j \left(\frac{C_{ij}}{L_{ij}} \right) \right]}{\max_j \left(\frac{C_{ij}}{L_{ij}} \right)}, \qquad [14]$$

where C_{ij} is the number of crashes for section j in bin i and L_{ij} is the length of section j in bin i. The F_i correction factor ensures that the maximum number of crashes remains constant (equal to the minimum of maximum section crash rates) across the various bins, which is by definition what an exposure measure is. The correction factor is also equal to

$$F_i = aV_i^b , \qquad [15]$$

where V_i is the mean AADT volume across all observations j in bin i and α and β are model coefficients. By solving Equation 14 and Equation 15 simultaneously we derive

$$CR = F_i \times \max_j \left(\frac{C_{ij}}{L_{ij}} \right) = aV_i^b \times \max_j \left(\frac{C_{ij}}{L_{ij}} \right) = a \times \frac{C_i}{L_i V_i^{-b}} = \frac{\text{Crashes}}{\text{Length} \times \text{AADT}^p} . \qquad [16]$$

Consequently, β is equivalent to $-p$ and can be solved for by fitting a regression line to the logarithmic transformation of Equation 15 as

$$\ln(F_i) = \ln(a) + b \ln(V_i) . \qquad [17]$$

After applying a least squared fit to the data, the model residual errors were tested for normality. There was insufficient evidence to reject the data normality assumption and thus the regression results are not biased.

Computation of Crash Rate

Once the exponent of the AADT was estimated, the crash rate was computed for each of the 186 study sections. A linear regression model in a single independent variable of the form

$$\ln(CR) = b_0 + b_1 L_1 \qquad [18]$$

was fit to the data. In order to satisfy data normality, the data were sorted based on L_1 and aggregated into equally sized bins of eight observations. It should be noted that the typical approach to binning is to use equal intervals for binning as opposed to equally sized bins. This unique data aggregation approach is equivalent to considering a longer analysis period (in this case, considering an analysis period of 8×5=40 years). The data aggregation increases the level of exposure and thus reduces the number of zero crash observations (in this case, zero observations are reduced), given that it is highly unlikely to have no crashes over a 40-year period. For each bin, the average section length (L_1) and crash rate (*CR*) were computed. As demonstrated later in the report, there was insufficient evidence to reject the data error normality and homoscedasticity assumption for the log-transformed data and thus a least squares GLM could be applied to the data.

A robust linear regression was applied to the data to derive the model parameters and remove outlier data. This procedure dampens the effect of observations that would be highly influential if least squares were used (Montgomery et al., 2001). The robust linear regression fit uses an iteratively re-weighted least squares algorithm, with the weights at each iteration calculated by applying the bisquare function to the residuals from the previous iteration. This Matlab algorithm gives lower weight to points that appear to be outliers. Data that should be disregarded are given a weight of zero. Consequently, the regression model is less sensitive to outliers in the data as compared with ordinary least squares regression. Data observations with zero weights were removed from the analysis (in this case, a single observation was removed).

Similarly, a regression model was fit to the data considering the independent variable as the distance to the first intersection. A similar robust regression was applied to the data to derive the model intercept and slope. Given that the intercept confidence limits included the value of intercept of the first model, the intercept was kept constant in both models. A regression was then performed to estimate the optimum slope. In summary, the final models that were developed are of the form

$$C = \frac{(365 \times 5)^p}{10^6} \times \exp\left(b_0 + b_1 L_1 + \ln(L_2) + p\ln(V)\right), \text{ or} \qquad [19]$$

$$C = \frac{(365 \times 5)^p}{10^6} \exp\left(b_0\right) \times \exp\left(b_1 L_1\right) \times L_2 V^p = g \times \exp\left(b_1 L_1\right) \times L_2 V^p . \qquad [20]$$

The number of crashes in a single year (*C'*) can be computed by adjusting the model intercept by the (p-1)×ln(5) as

$$C' = \frac{365^p}{10^6} \times \exp\left(b_0 + (p-1)\ln 5 + b_1 L_1 + \ln(L_2) + p\ln(V)\right). \qquad [21]$$

The crash rate for a single year in million vehicle-miles where the traffic volume is raised to the exponent p (*CR'*) can be computed as

$$CR' = \exp\left(b_0 + (p-1)\ln 5 + b_1 L_1\right). \qquad [22]$$

The crash rate in vehicle miles traveled (VMT) considering an exponent of 1.0 (*CR"*) is computed as

$$CR'' = \exp\left(b_0 + (p-1)\ln 5 + b_1 L_1\right) \times 1.6\left(365V\right)^{p-1}, \text{ or}$$

$$CR'' = \exp\left(b_0 + \ln 1.6 + (p-1)\ln 5 + (p-1)\ln 365 + b_1 L_1 + (p-1)\ln V\right) \qquad [23]$$

The approach can be summarized as follows:

1. Consider the use of an exponential function. This function ensures that the number of crashes equals zero when the exposure is zero; that the number of crashes are always positive; and that the model reverts to a linear function after performing a logarithmic transformation.
2. Sort and aggregate the data based on the AADT using a variable bin size while ensuring that the second independent variable remains constant across the various bins.
3. Compute crash adjustment factors to normalize the maximum number of crashes across the various bins.
4. Perform a logarithmic transformation on the crash adjustment factors to compute the AADT exponent using an LRM while ensuring that the data satisfy the LRM assumptions of normality and homoscedasticity.
5. Compute crash rates using the AADT exponent that was computed earlier and then sort and aggregate the crash rate data based on the second independent variable using an equally sized bin structure (equal number of observations in each bin).
6. Compute the average dependent and independent variables for each bin.
7. Perform a logarithmic transformation of the data and ensure normality and homoscedasticity to develop the final crash prediction model.

The proposed approach was tested and validated using data from 186 access road sections in the state of Virginia. The approach was demonstrated to be superior to traditional negative binomial models because it is not influenced (through data aggregation) by the prevalence of the large number of zero observations that are typical of crash data.

Applying the Models

The results presented in this section consider the distance to the first access road to be measured from the gore of the freeway off-ramp to the centerline of the access road. An additional analysis considering the access distance to be measured from the acceleration lane taper to the access road centerline is presented in Appendix C.

Poisson or Negative Binomial Model Approach

The Poisson models were fit to the data to derive

$$\frac{C}{L_2} = \frac{1825^p}{1000000} \times \exp\left(b_0 + b_1 L_1 + p \ln(V)\right) \qquad [24]$$

The estimated model is shown in Equation 25 and the parameters for both models with and without zeros are shown in Table 8.

$$\frac{C}{L_2} = \left(\frac{(1825)^{0.9161}}{10^6}\right) \times \exp\left(2.1382 - 6.2107 \times L_1 + 0.9161 \times \ln(V)\right) \qquad [25]$$

Table 8. Summary Results of Poisson Regression

a. Analysis of Parameter Estimates						
	With Zeros			**Without Zeros**		
Parameter	DF	Estimate	Pr > Chi2	DF	Estimate	Pr > Chi2
Intercept	1	2.1382	<.0001	1	2.4439	<.0001
L_1	1	-6.2107	<.0001	1	-5.5794	<.0001
Ln(V)	0	0.9161		0	0.8613	
Scale	0	1.0000		0	1.0000	
b. Criteria For Assessing Goodness of Fit						
	With Zeros			**Without Zeros**		
Criterion	DF	Value	Value/DF	DF	Value	Value/DF
Deviance	184	33803.4292	184.7182	145	28277.6521	196.3726
Scaled Deviance	184	33803.4292	184.7182	145	28277.6521	196.3726
Pearson Chi-Square	184	48333.9089	264.1197	145	37822.7605	262.6581
Scaled Pearson X2	184	48333.9089	264.1197	145	37822.7605	262.6581
Log Likelihood		77562.4703			80325.3589	

Assessing the goodness of fit for the model, one can observe that the Poisson regression model suffers from over-dispersion (184.7182>1 for data that include the zero and 196.3726>1 for data without the zeros).

Consequently, the Negative Binomial model was fit to the data in Equation 4 and the model parameters are summarized in Table 9. All of the parameters are found to be significant (p values <0.0001).

Table 9. Summary Results for Negative Binomial Regression

a. Analysis of Parameter Estimates						
	With Zeros			**Without Zeros**		
Parameter	DF	Estimate	Pr > Chi2	DF	Estimate	Pr > Chi2
Intercept	1	2.3216	<.0001	1	2.6771	<.0001
L_1	1	-3.7584	<.0001	1	-3.3318	<.0001
Ln(V)	0	0.7903		0	0.7256	
Dispersion	1	2.7010		1	1.3757	
b. Criteria For Assessing Goodness of Fit						
	With Zeros			**Without Zeros**		
Criterion	DF	Value	Value/DF	DF	Value	Value/DF
Deviance	184	220.7602	1.2063	145	172.9626	1.2011
Scaled Deviance	184	220.7602	1.2063	145	172.9626	1.2011
Pearson Chi-Square	184	158.9742	0.8687	145	181.6395	1.2614
Scaled Pearson X2	184	158.9742	0.8687	145	181.6395	1.2614
Log Likelihood		93991.7767			94076.3488	

Assessing the goodness of fit for the model, deviance divided by the degrees of freedom is quite small (1.2063 and 1.2011) as shown in the table, showing a better model than the Poisson model.

The second approach is the modified Poisson regression by adding an over-dispersion parameter to the model to inflate the variability of the estimators. The model was fit to the data in Equation 24 and the parameters are summarized in Table 10. All of the parameters are found to be significant (p values of <0.0001) as shown in Table 10.

Table 10. Summary Results of Modified Poisson Regression

a. Analysis of Parameter Estimates						
	With Zeros			**Without Zeros**		
Parameter	DF	Estimate	Pr > Chi2	DF	Estimate	Pr > Chi2
Intercept	1	2.1382	<.0001	1	2.4439	<.0001
L_1	1	-6.2107	<.0001	1	-5.5794	<.0001
Ln(V)	0	0.9161		0	0.8613	
Scale	0	13.5907		0	14.0127	
b. Criteria For Assessing Goodness of Fit						
	With Zeros			**Without Zeros**		
Criterion	DF	Value	Value/DF	DF	Value	Value/DF
Deviance	184	33801.4567	184.7074	145	28275.418	196.3571
Scaled Deviance	184	183.0000	1.0000	145	144.0000	1.0000
Pearson Chi-Square	184	48330.3086	264.1000	145	37819.52	262.6356
Scaled Pearson X2	184	261.6587	1.4298	145	192.6059	1.3375
Log Likelihood		419.9479			409.1042	

The results of the test for the coincidence of the two regression models' test is presented in Table 11. It shows, considering a significance level (α =0.05), that none of the variables are significant variables. Consequently, the full model does not need to be modified.

Table 11. Summary of Significance of Other Variables

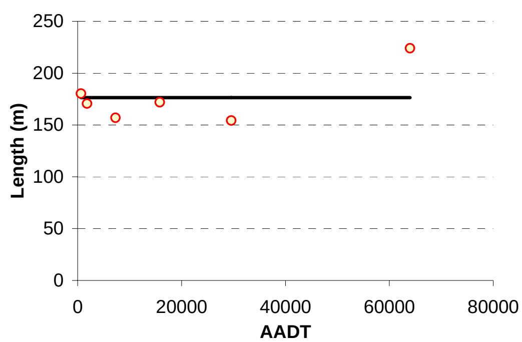
Condition		B_0	B_1	**P-value**
Median?	Yes	3.0221	-4.8858	0.0830
	No	3.7713	-9.1647	
Acceleration. Lane?	Yes	3.6943	-8.6222	0.369
	No	3.2653	-5.7755	
Urban?	Yes	3.2934	-6.6971	0.446
	No	3.5672	-7.1335	
Left Turn Storage?	Yes	3.0607	-5.147	0.269
	No	3.5729	-7.8569	
Number of Legs	1	3.3966	-6.9097	0.803
	>1	3.61	-7.3964	
Signalized?	Yes	3.0629	-5.8839	0.177
	No	3.5578	-7.2245	
Number of Lanes	2	3.4867	-6.7333	0.846
	>2	3.3728	-6.7784	
Type of First Access	Yes	3.7648	-8.0815	0.0525
	No	3.0255	-5.9062	

The model can be cast as

$$D = \begin{cases} 1, & CR = \exp\left((b_0 + b_2) + (b_1 + b_3)L_1\right) \\ 0, & CR = \exp\left(b_0 + b_1 L_1\right) \end{cases} \qquad [26]$$

Linear Regression Modeling Approach

As was mentioned, it was important to normalize one of the variables while analyzing the second variable. In order to estimate the volume exponent, the data were sorted based on their AADT values and aggregated using variable bin sizes to ensure that the L_1 variable remained constant across the various bins, as illustrated in Figure 25. The figure demonstrates that by performing a linear regression of L_1 against V, the slope of the line is insignificant ($p > 0.05$), and thus there is insufficient evidence to conclude that the L_1 variable varies across the aggregated data.



Regression Statistics	
R^2	0.509
R^2 (Adj)	0.386
Obs.	6

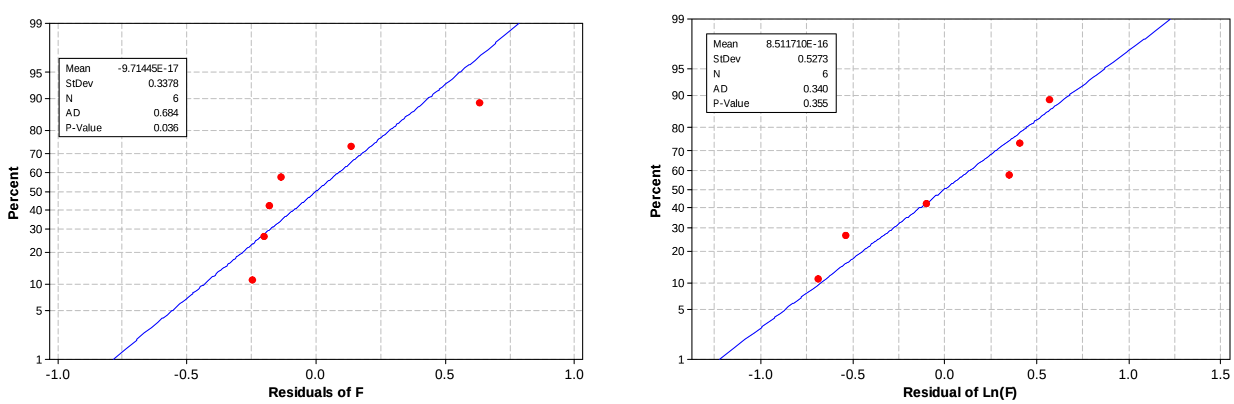
ANOVA

	df	*SS*	*MS*	*F*	*ignificance F*
Reg.	1	17608.38	17608.38	4.14	0.11
Residual	4	17003.46	4250.86		
Total	5	34611.84			

	Coeff.	*SE*	*t Stat*	*P-value*	*Low 95%*	*Up 95%*
Intercept	530.039	35.820	14.80	0.00	430.59	629.49
AADT	0.002	0.001	2.04	0.11	0.00	0.01

Figure 25. Effect of Access Section Length within AADT Binning

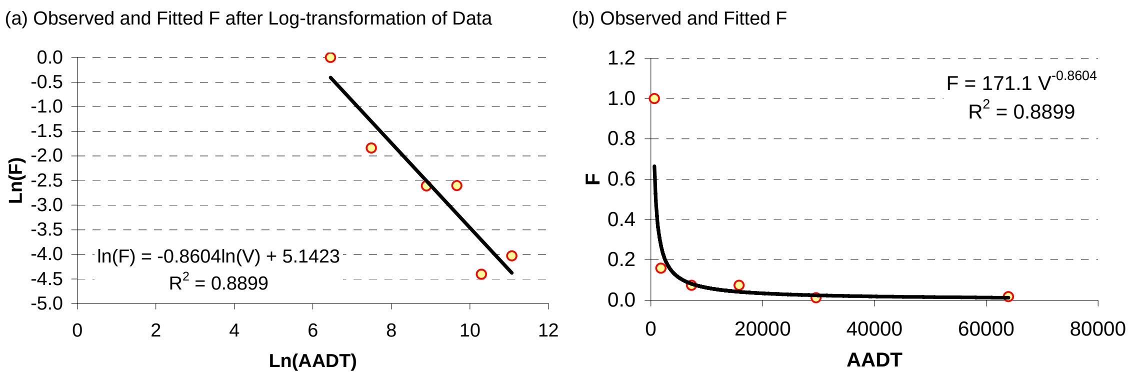
After applying a least squared fit to the data, the model residual errors were tested for normality. While in the case of the original non-transformed data the residual error did not pass the normality test, the log-transformed residual errors did pass the test ($p = 0.355$), as illustrated in Figure 26.



(a) Normality Test on F_i (b) Normality Test on Log-transformed F_i

Figure 26. Test of Normality of AADT Adjustment Factors

A least squares LRM was then fit to the log-transformed data producing an R^2 of 0.89, as illustrated in Figure 27. The model was statistically significant ($p < 0.005$) and both the intercept and slope coefficients were significant ($p = 0.02$ and 0.00, respectively). Consequently, the exponent of the AADT for utilization in the exposure measure is 0.86, which is very similar to what was derived from the negative binomial fit to the data ($p = 0.80$). If we consider distances in miles the exponent is 0.8695, which is very similar to what was derived from the negative binomial fit to the data (p = 0.88).



Regression Statistics	
R^2	0.890
R^2 (Adj)	0.862
Obs.	6

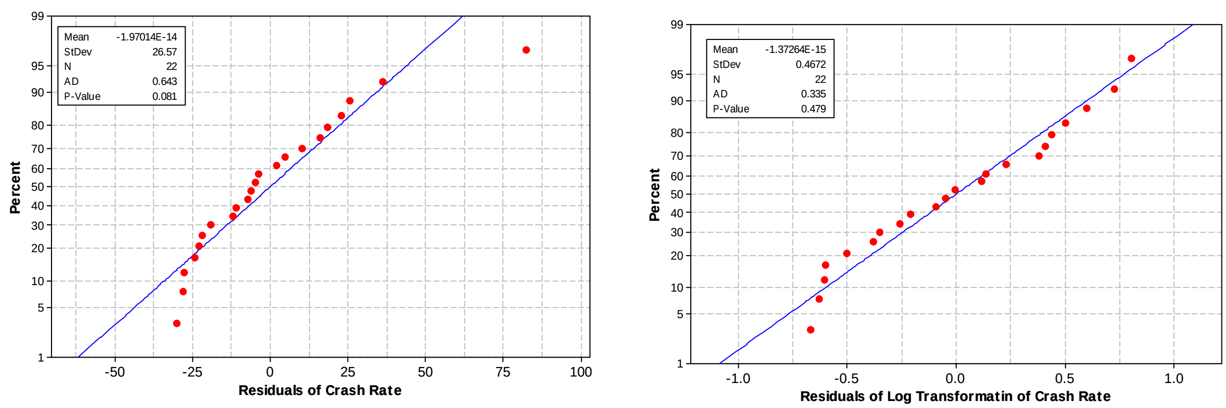
ANOVA

	df	*SS*	*MS*	*F*	*Sig. F*
Reg.	1	11.24	11.24	32.33	0.00
Residual	4	1.39	0.35		
Total	5	12.63			

	Coeff.	*SE*	*t Stat*	*P-value*	*Low 95%*	*Up 95%*
Intercept	5.1423	1.38	3.73	0.02	1.31	8.97
ln(AADT)	-0.8604	0.15	-5.69	0.00	-1.28	-0.44

Figure 27. Computation of Exposure Measures

Once the exponent of the AADT was estimated, the crash rate was computed for each of the 186 study sections. A linear regression model was fit to the data. As demonstrated in Figure 28, there was insufficient evidence to reject the data error normality and homoscedasticity assumption for the log-transformed data (p=0.479) and thus a least squares GLM could be applied to the data.

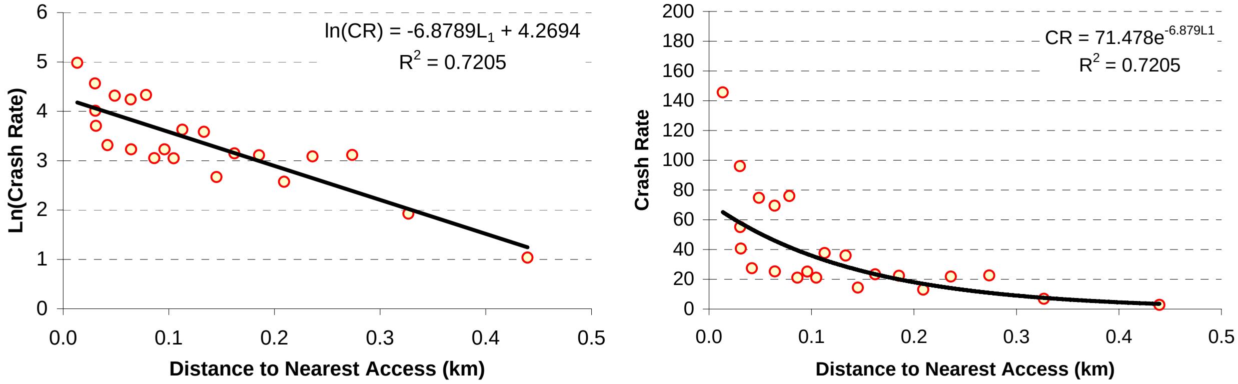


(a) Normality Test on Crash Rate

(b) Normality Test on Log-transformed Crash Rate

Figure 28. Test of Normality for Crash Rate Data

A robust linear regression was applied to the data to derive the model parameters and remove outlier data. The results of the analysis demonstrate a statistically significant model (F=51.56 and p<0.0005) with an R^2 of 0.72.(Figure 29) The intercept and L_1 coefficients are statistically significant (p<0.0005 and p<0.0005, respectively) with values of 4.269 and -6.879, respectively. In the case of distances in miles the intercept and L_1 coefficients are statistically significant (p<0.0005 and p<0.0005, respectively) with values of 4.96 and -9.674, respectively).



Regression Statistics	
Multiple R	0.849
R^2	0.721
R^2 (Adj)	0.707
SE	0.479
Obs.	22

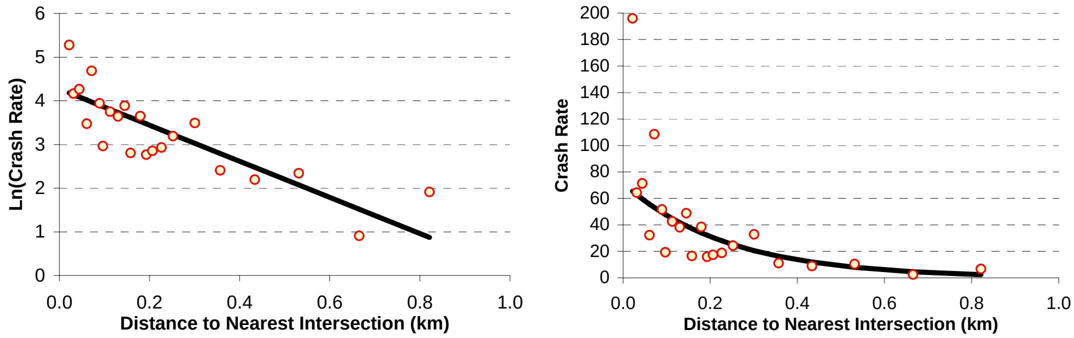
ANOVA

	df	*SS*	*MS*	*F*	*Sig. F*
Regressior	1	11.819	11.819	51.563	0.000
Residual	20	4.584	0.229		
Total	21	16.403			

	Coeff.	*SE*	*t Stat*	*P-value*	*Low 95%*	*Up 95%*
Intercept	4.269	0.163	26.222	0.000	3.930	4.609
L_1	-6.879	0.958	-7.181	0.000	-8.877	-4.881

Figure 29. Crash Prediction Model Considering Nearest Access Point

Similarly, a regression model was fit to the data considering the independent variable as the distance to the first intersection. A similar robust regression was applied to the data to derive the model intercept and slope. Given that the intercept confidence limits included the value of intercept of the first model, the intercept was kept constant in both models. A regression was then performed to estimate the optimum slope. The model is significant (F=111.44 and p<0.0005) with an R^2 of 0.85. The slope of the line is significant (p<0.0005) with a value of -4.135. In the case of distances in miles the slope of the line is significant (p<0.0005) with a value of -9.656 (Figure 30).



Regression Statistics	
Multiple R	0.921
R^2	0.848
R^2 (Adj)	0.801
SE	0.558
Obs.	22

ANOVA

	df	*SS*	*MS*	*F*	*Sig. F*
Regressior	1	36.552	36.552	117.436	0.000
Residual	21	6.536	0.311		
Total	22	43.088			

	Coeff.	*SE*	*t Stat*	*P-value*	*Low 95%*	*Up 95%*
Intercept	4.269	#N/A	#N/A	#N/A	#N/A	#N/A
L_2	-4.135	0.382	-10.837	0.000	-4.928	-3.341

Figure 30. Crash Prediction Model Considering Nearest Intersection

The crash rate in VMT considering an exponent of 1.0 (*CR”*) can computed as

$$CR'' = \exp\left(b_0 + (p - 1)\ln 5 + b_1 L_1\right) \times 1.6(365V)^{p-1}, \text{ or} \quad [27]$$

$$CR'' = \exp\left(b_0 + \ln 1.6 + (p - 1)\ln 5 + (p - 1)\ln 365 + b_1 L_1 + (p - 1)\ln V\right) \quad [28]$$

DISCUSSION

Poisson or Negative Binomial Model Approach

In an attempt to validate the model, the AADT and access road spacing parameters for the 186 study sections were input into the modified Poisson regression model. A comparison between the observed and estimated crashes revealed a reasonable level of correlation (Pearson correlation coefficient 0.25), as illustrated in Figure 31.

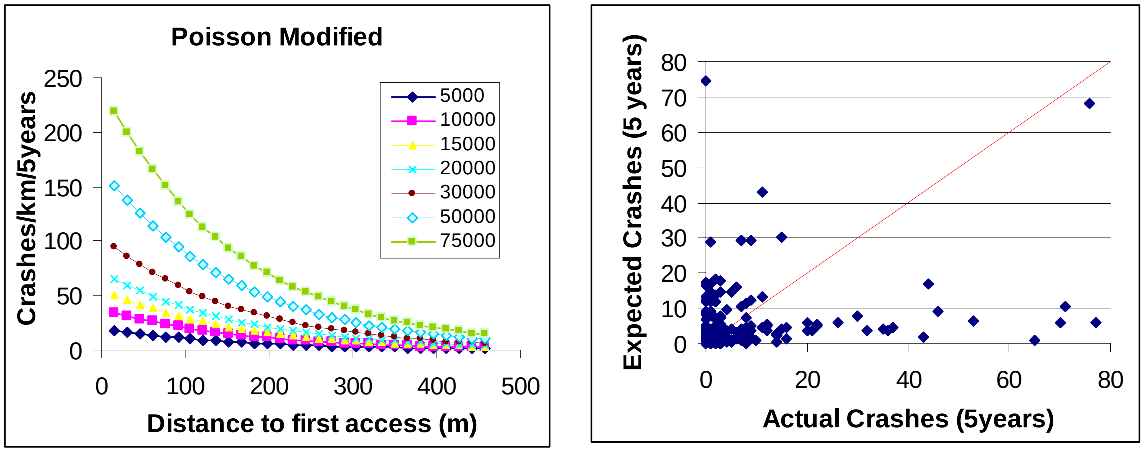


Figure 31. Variation in the Expected Number of Crashes as Function to the Distance to the First Access and Comparison of Actual and Expected Crashes

The crash predictions using the NB and Poisson model revealed that the Poisson model produced significantly better crash predictions, as illustrated in Figure 32. It should be noted that the NB model significantly underestimated the expected number of crashes.

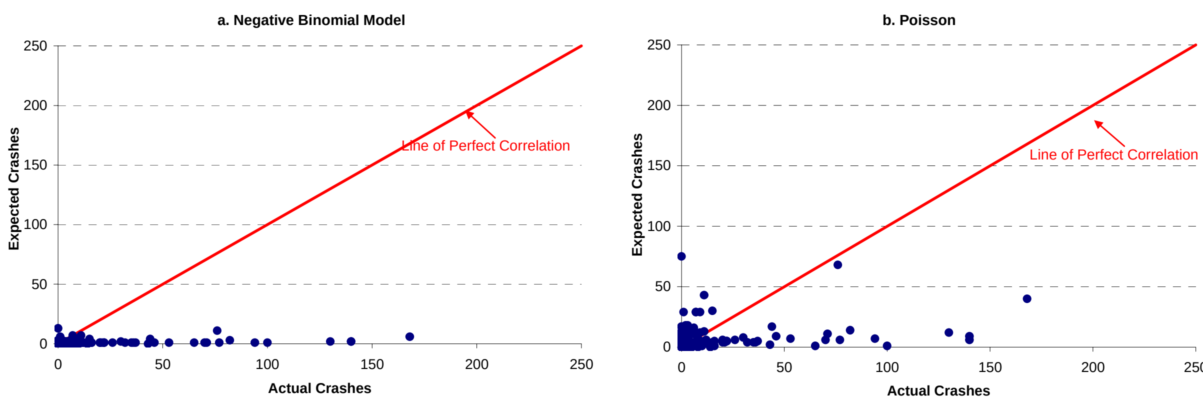


Figure 32. Comparison of Negative Binomial and Poisson Model Estimates

A number of researchers have argued for the need to calibrate the exposure AADT exponent. Consequently, a sensitivity analysis was conducted to study the impact of alternative exponents on the CPM predictions. Exponent values ranging from 0.6 to 1.2 were evaluated based on values reported in the literature (Qin et al., 2004), as illustrated in Figure 33. The results clearly indicate that the crash predictions increase as the exponent increases; however, the variation in crash predictions as a function of V and L_1 remains similar. The results demonstrate the calibration of the exposure parameter might not be critical for comparison purposes but is critical when estimating the number of crashes.

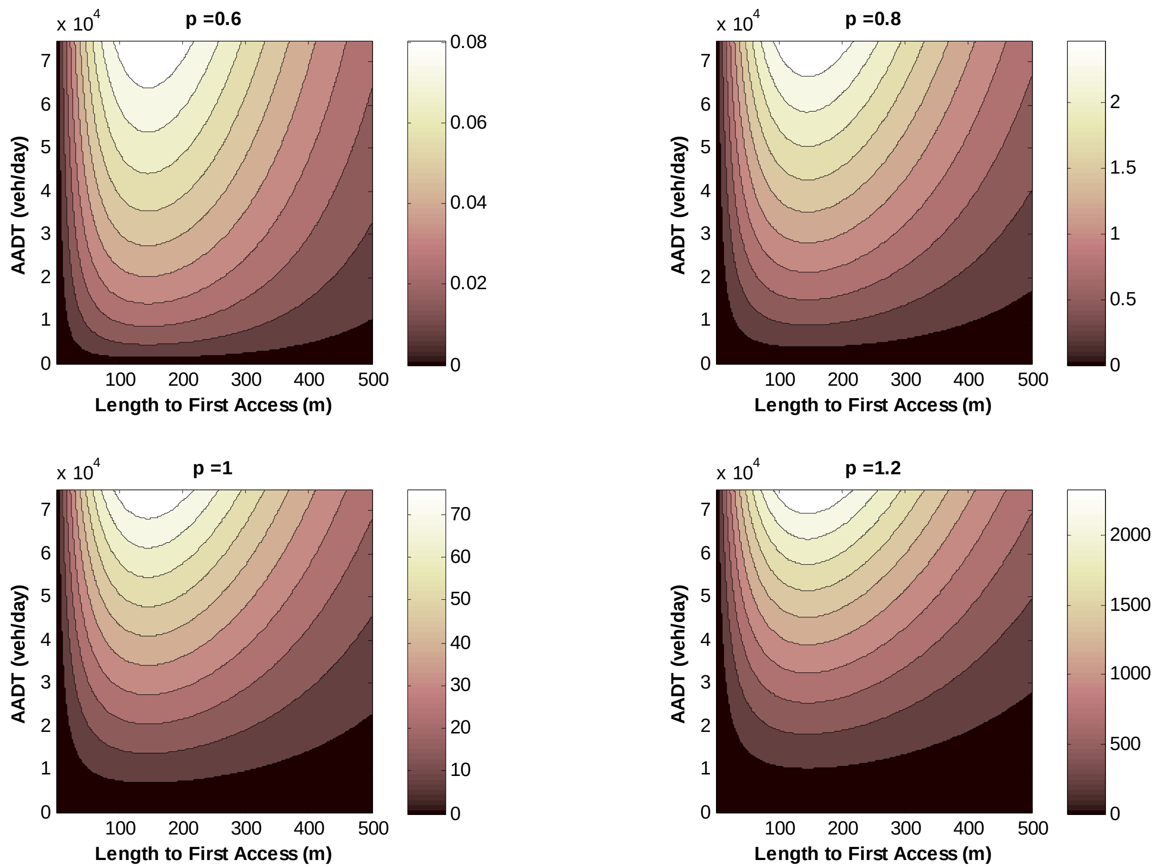


Figure 33. Variation in Expected Crashes as a Function of AADT Exponent

It should be noted that for a constant AADT the expected crashes initially increase as the spacing between the freeway ramp and the first access road section increases before decreasing again, as demonstrated in Figure 33.

Figure 34 demonstrates that the maximum expected crashes occurs at an access road spacing of approximately 150 m (500 ft). The observed behavior might appear to be counterintuitive at first glance; however, it can be explained by the fact that as the study section increases, the expected number of crashes per unit distance decreases (as illustrated in Figure 35 and Figure 36) while the level of exposure increases. Initially, the rate of increase in the level of exposure exceeds the rate of decrease in the crash rate, producing an increase in the number of crashes. Consequently, decisions should be made using either a crash rate or the expected number of crashes per unit distance, as illustrated in Figure 36.

The expected number of crashes for an access road spacing of 100 ft and 300 ft is highlighted to demonstrate the current criteria for access road spacing.

The average number of crashes across all 186 study sections was 2.45 crashes/year with an average AADT of 19,456 and an average access road spacing of 169 m (550 ft). The expected number of crashes derived from the model for the same AADT and access road spacing is estimated by the linear regression model at 2.43 crashes/year (highlighted in Figure 34) and thus demonstrating the validity of the model results. Alternatively, the negative binomial model predicts 0.2 crashes/year, which is significantly less than the field-observed crashes.

L (m)	L (ft)	AADT (veh/day)										
		5000	10000	15000	20000	25000	30000	35000	40000	45000	50000	75000
0.0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15.3	50	0.19	0.35	0.49	0.63	0.76	0.89	1.02	1.14	1.26	1.38	1.96
30.6	100	0.34	0.62	0.88	1.13	1.37	1.61	1.83	2.06	2.28	2.49	3.53
45.9	150	0.46	0.84	1.20	1.53	1.86	2.17	2.48	2.78	3.08	3.37	4.77
61.2	200	0.56	1.01	1.44	1.84	2.23	2.61	2.98	3.34	3.69	4.04	5.73
76.5	250	0.63	1.14	1.62	2.07	2.51	2.93	3.35	3.76	4.16	4.55	6.45
91.7	300	0.68	1.23	1.75	2.24	2.71	3.17	3.62	4.06	4.49	4.92	6.97
107.0	350	0.71	1.29	1.83	2.35	2.85	3.33	3.80	4.26	4.72	5.17	7.32
122.3	400	0.73	1.33	1.89	2.42	2.93	3.43	3.91	4.39	4.86	5.32	7.54
137.6	450	0.74	1.35	1.91	2.45	2.97	3.47	3.96	4.45	4.92	5.39	7.63
152.9	500	0.74	1.35	1.91	2.45	2.97	3.47	3.97	4.45	4.92	5.39	7.64
168.2	550	0.74	1.34	1.89	2.43	2.94	3.44	3.93	4.41	4.88	5.34	7.57
183.5	600	0.72	1.31	1.86	2.38	2.89	3.38	3.86	4.33	4.79	5.24	7.43
198.8	650	0.71	1.28	1.82	2.33	2.82	3.30	3.76	4.22	4.67	5.12	7.25
214.1	700	0.68	1.24	1.76	2.26	2.73	3.20	3.65	4.09	4.53	4.96	7.03
229.4	750	0.66	1.20	1.70	2.18	2.64	3.08	3.52	3.95	4.37	4.79	6.78
244.6	800	0.63	1.15	1.63	2.09	2.53	2.96	3.38	3.79	4.20	4.60	6.52
259.9	850	0.61	1.10	1.56	2.00	2.42	2.83	3.24	3.63	4.02	4.40	6.23
275.2	900	0.58	1.05	1.49	1.91	2.31	2.70	3.09	3.46	3.83	4.19	5.94
290.5	950	0.55	1.00	1.41	1.81	2.20	2.57	2.93	3.29	3.64	3.99	5.65
305.8	1000	0.52	0.95	1.34	1.72	2.08	2.43	2.78	3.12	3.45	3.78	5.36
321.1	1050	0.49	0.89	1.27	1.62	1.97	2.30	2.63	2.95	3.26	3.57	5.06
336.4	1100	0.46	0.84	1.20	1.53	1.86	2.17	2.48	2.78	3.08	3.37	4.78
351.7	1150	0.44	0.79	1.13	1.44	1.75	2.04	2.33	2.62	2.90	3.17	4.50
367.0	1200	0.41	0.75	1.06	1.36	1.64	1.92	2.19	2.46	2.72	2.98	4.23
382.3	1250	0.39	0.70	0.99	1.27	1.54	1.80	2.06	2.31	2.55	2.80	3.96
397.5	1300	0.36	0.66	0.93	1.19	1.44	1.69	1.93	2.16	2.39	2.62	3.71
412.8	1350	0.34	0.61	0.87	1.11	1.35	1.58	1.80	2.02	2.24	2.45	3.47
428.1	1400	0.32	0.57	0.81	1.04	1.26	1.47	1.68	1.89	2.09	2.29	3.24
443.4	1450	0.29	0.53	0.76	0.97	1.17	1.37	1.57	1.76	1.95	2.13	3.02
458.7	1500	0.27	0.50	0.71	0.90	1.09	1.28	1.46	1.64	1.81	1.99	2.82

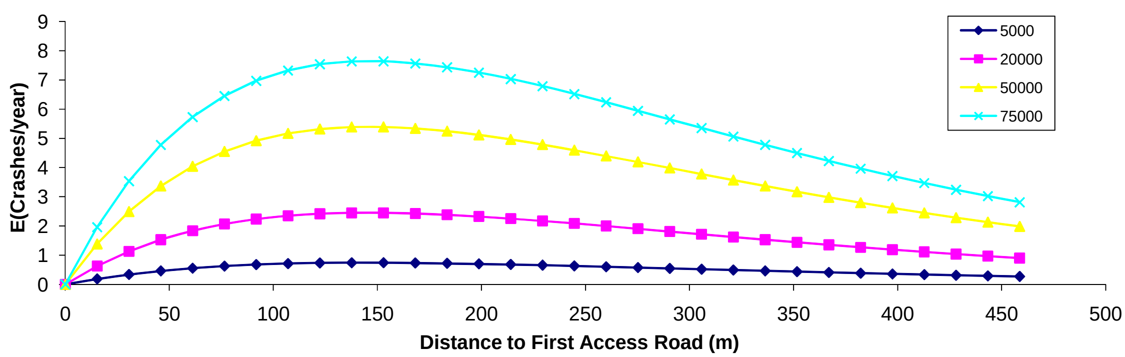


Figure 34: Variation in the Expected Number of Yearly Crashes As a Function of the Access Section Length and AADT

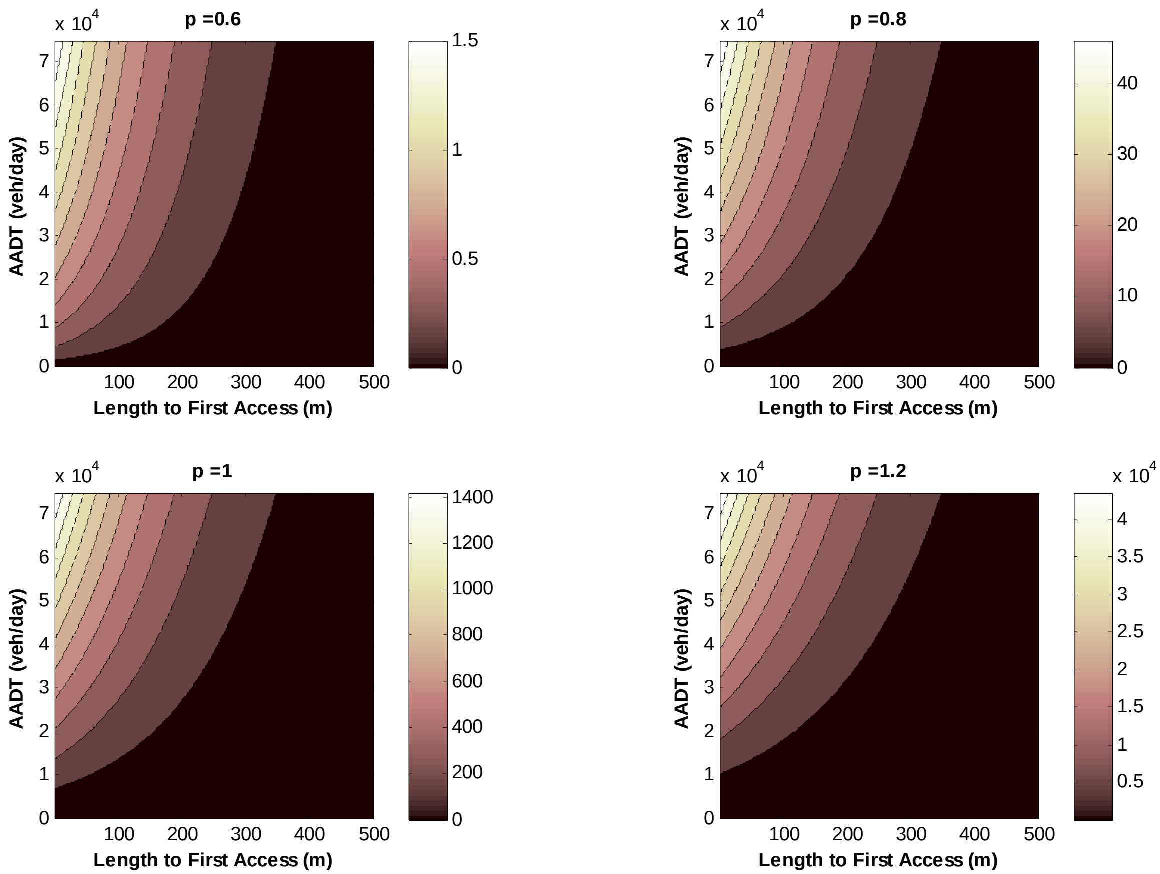


Figure 35. Variation in Expected Crashes/Km As a Function of AADT Exponent

L (m)	L (ft)	AADT (veh/day)										
		5000	10000	15000	20000	25000	30000	35000	40000	45000	50000	75000
0.0	0											
15.3	50	19.98	36.28	51.42	65.86	79.80	93.35	106.59	119.57	132.32	144.88	205.35
30.6	100	17.99	32.67	46.30	59.31	71.86	84.06	95.99	107.67	119.15	130.46	184.92
45.9	150	16.20	29.42	41.69	53.40	64.71	75.70	86.43	96.95	107.29	117.47	166.51
61.2	200	14.59	26.49	37.55	48.09	58.27	68.16	77.83	87.30	96.62	105.78	149.94
76.5	250	13.14	23.85	33.81	43.30	52.47	61.38	70.08	78.62	87.00	95.25	135.02
91.7	300	11.83	21.48	30.44	38.99	47.25	55.27	63.11	70.79	78.34	85.77	121.58
107.0	350	10.65	19.34	27.41	35.11	42.54	49.77	56.83	63.75	70.54	77.24	109.48
122.3	400	9.59	17.42	24.69	31.62	38.31	44.82	51.17	57.40	63.52	69.55	98.58
137.6	450	8.64	15.68	22.23	28.47	34.50	40.36	46.08	51.69	57.20	62.63	88.77
152.9	500	7.78	14.12	20.02	25.64	31.06	36.34	41.49	46.54	51.51	56.39	79.94
168.2	550	7.00	12.72	18.02	23.09	27.97	32.72	37.36	41.91	46.38	50.78	71.98
183.5	600	6.31	11.45	16.23	20.79	25.19	29.47	33.64	37.74	41.77	45.73	64.82
198.8	650	5.68	10.31	14.61	18.72	22.68	26.53	30.30	33.98	37.61	41.18	58.37
214.1	700	5.11	9.28	13.16	16.86	20.42	23.89	27.28	30.60	33.87	37.08	52.56
229.4	750	4.61	8.36	11.85	15.18	18.39	21.51	24.57	27.56	30.49	33.39	47.33
244.6	800	4.15	7.53	10.67	13.67	16.56	19.37	22.12	24.81	27.46	30.07	42.62
259.9	850	3.73	6.78	9.61	12.31	14.91	17.44	19.92	22.34	24.73	27.07	38.37
275.2	900	3.36	6.10	8.65	11.08	13.43	15.71	17.94	20.12	22.27	24.38	34.55
290.5	950	3.03	5.50	7.79	9.98	12.09	14.15	16.15	18.12	20.05	21.95	31.12
305.8	1000	2.73	4.95	7.02	8.99	10.89	12.74	14.54	16.31	18.05	19.77	28.02
321.1	1050	2.46	4.46	6.32	8.09	9.80	11.47	13.10	14.69	16.26	17.80	25.23
336.4	1100	2.21	4.01	5.69	7.29	8.83	10.33	11.79	13.23	14.64	16.03	22.72
351.7	1150	1.99	3.61	5.12	6.56	7.95	9.30	10.62	11.91	13.18	14.43	20.46
367.0	1200	1.79	3.25	4.61	5.91	7.16	8.37	9.56	10.73	11.87	13.00	18.42
382.3	1250	1.61	2.93	4.15	5.32	6.45	7.54	8.61	9.66	10.69	11.70	16.59
397.5	1300	1.45	2.64	3.74	4.79	5.80	6.79	7.75	8.70	9.63	10.54	14.94
412.8	1350	1.31	2.38	3.37	4.31	5.23	6.11	6.98	7.83	8.67	9.49	13.45
428.1	1400	1.18	2.14	3.03	3.88	4.71	5.51	6.29	7.05	7.80	8.55	12.11
443.4	1450	1.06	1.93	2.73	3.50	4.24	4.96	5.66	6.35	7.03	7.69	10.91
458.7	1500	0.96	1.73	2.46	3.15	3.82	4.46	5.10	5.72	6.33	6.93	9.82

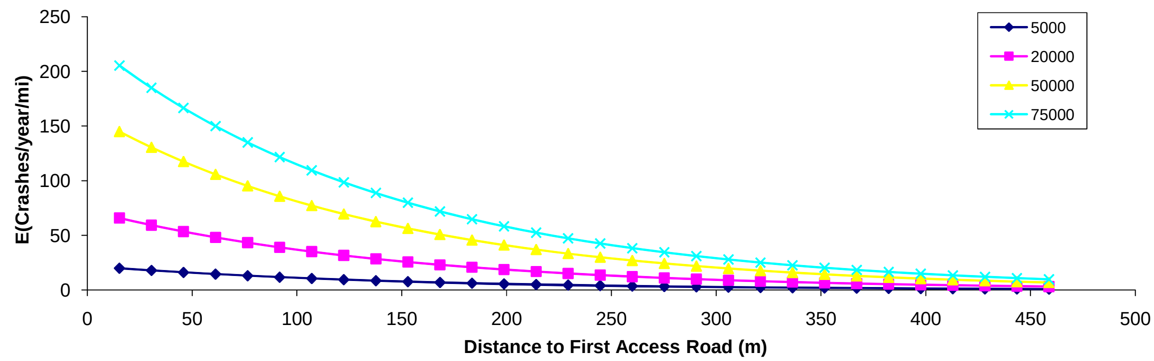


Figure 36. Variation in the Expected Number of Yearly Crashes per Mile As a Function of the Access Section Length and AADT

The 95% confidence intervals were computed as shown in Figure 37 and Figure 38.

L (m)	L (ft)	AADT (veh/day)										
		5000	10000	15000	20000	25000	30000	35000	40000	45000	50000	75000
0.0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15.3	50	1.06	1.93	2.73	3.50	4.24	4.97	5.67	6.36	7.04	7.71	10.92
30.6	100	1.91	3.47	4.93	6.31	7.64	8.94	10.21	11.45	12.67	13.88	19.67
45.9	150	2.59	4.69	6.65	8.52	10.32	12.08	13.79	15.47	17.12	18.74	26.57
61.2	200	3.10	5.64	7.99	10.23	12.40	14.50	16.56	18.57	20.55	22.51	31.90
76.5	250	3.49	6.34	8.99	11.52	13.95	16.32	18.64	20.91	23.14	25.33	35.91
91.7	300	3.78	6.85	9.72	12.44	15.08	17.64	20.14	22.59	25.00	27.37	38.80
107.0	350	3.97	7.20	10.21	13.07	15.84	18.53	21.16	23.73	26.26	28.76	40.76
122.3	400	4.08	7.41	10.50	13.45	16.30	19.07	21.77	24.42	27.03	29.59	41.95
137.6	450	4.13	7.51	10.64	13.63	16.51	19.32	22.06	24.74	27.38	29.98	42.49
152.9	500	4.14	7.51	10.65	13.64	16.52	19.33	22.07	24.76	27.40	29.99	42.52
168.2	550	4.10	7.44	10.54	13.51	16.36	19.14	21.86	24.52	27.14	29.71	42.11
183.5	600	4.03	7.31	10.36	13.27	16.08	18.81	21.47	24.09	26.66	29.19	41.37
198.8	650	3.93	7.13	10.11	12.94	15.68	18.35	20.95	23.50	26.00	28.47	40.36
214.1	700	3.81	6.91	9.80	12.55	15.21	17.79	20.31	22.79	25.22	27.61	39.13
229.4	750	3.67	6.67	9.45	12.11	14.67	17.16	19.60	21.98	24.33	26.64	37.76
244.6	800	3.53	6.41	9.08	11.63	14.09	16.49	18.82	21.12	23.37	25.59	36.27
259.9	850	3.38	6.13	8.69	11.13	13.48	15.77	18.01	20.20	22.36	24.48	34.70
275.2	900	3.22	5.84	8.28	10.61	12.86	15.04	17.17	19.26	21.32	23.34	33.08
290.5	950	3.06	5.55	7.87	10.08	12.22	14.29	16.32	18.31	20.26	22.18	31.44
305.8	1000	2.90	5.27	7.46	9.56	11.58	13.55	15.47	17.35	19.20	21.03	29.80
321.1	1050	2.74	4.98	7.06	9.04	10.95	12.81	14.63	16.41	18.16	19.88	28.18
336.4	1100	2.59	4.70	6.66	8.53	10.33	12.09	13.80	15.48	17.13	18.75	26.58
351.7	1150	2.44	4.42	6.27	8.03	9.73	11.38	12.99	14.57	16.13	17.66	25.03
367.0	1200	2.29	4.15	5.89	7.54	9.14	10.69	12.21	13.69	15.15	16.59	23.52
382.3	1250	2.15	3.90	5.52	7.07	8.57	10.03	11.45	12.84	14.21	15.56	22.06
397.5	1300	2.01	3.65	5.17	6.62	8.03	9.39	10.72	12.03	13.31	14.57	20.66
412.8	1350	1.88	3.41	4.84	6.19	7.51	8.78	10.03	11.25	12.45	13.63	19.32
428.1	1400	1.76	3.19	4.52	5.79	7.01	8.20	9.36	10.50	11.62	12.73	18.04
443.4	1450	1.64	2.97	4.21	5.40	6.54	7.65	8.73	9.80	10.84	11.87	16.82
458.7	1500	1.52	2.77	3.92	5.03	6.09	7.12	8.13	9.12	10.10	11.06	15.67

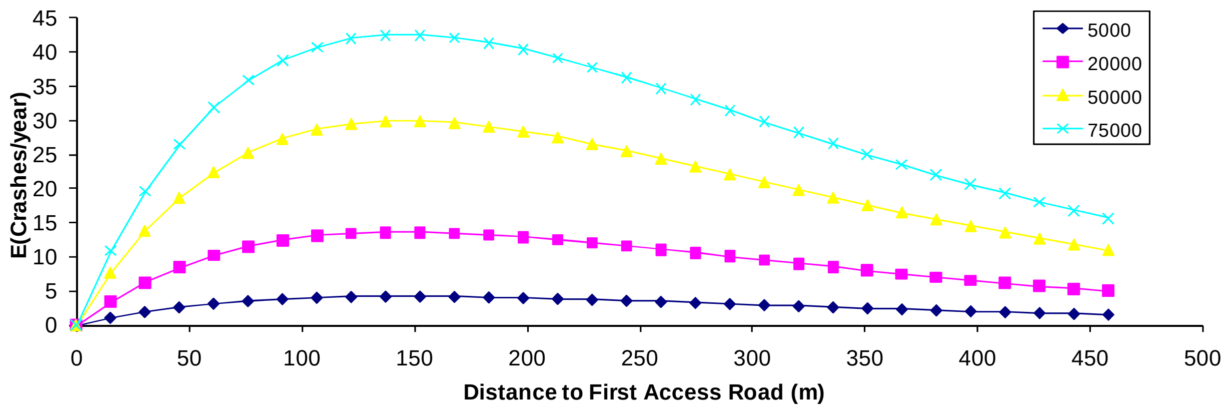


Figure 37. 95% Upper Confidence Limit of the Expected Number of Yearly Crashes As a Function of the Access Section Length and AADT

L (m)	L (ft)	AADT (veh/day)										
		5000	10000	15000	20000	25000	30000	35000	40000	45000	50000	75000
0.0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15.3	50	0.03	0.06	0.09	0.11	0.14	0.16	0.18	0.21	0.23	0.25	0.35
30.6	100	0.06	0.11	0.16	0.20	0.25	0.29	0.33	0.37	0.41	0.45	0.63
45.9	150	0.08	0.15	0.21	0.27	0.33	0.39	0.44	0.50	0.55	0.60	0.86
61.2	200	0.10	0.18	0.26	0.33	0.40	0.47	0.53	0.60	0.66	0.73	1.03
76.5	250	0.11	0.20	0.29	0.37	0.45	0.53	0.60	0.67	0.75	0.82	1.16
91.7	300	0.12	0.22	0.31	0.40	0.49	0.57	0.65	0.73	0.81	0.88	1.25
107.0	350	0.13	0.23	0.33	0.42	0.51	0.60	0.68	0.77	0.85	0.93	1.31
122.3	400	0.13	0.24	0.34	0.43	0.53	0.62	0.70	0.79	0.87	0.95	1.35
137.6	450	0.13	0.24	0.34	0.44	0.53	0.62	0.71	0.80	0.88	0.97	1.37
152.9	500	0.13	0.24	0.34	0.44	0.53	0.62	0.71	0.80	0.88	0.97	1.37
168.2	550	0.13	0.24	0.34	0.44	0.53	0.62	0.71	0.79	0.88	0.96	1.36
183.5	600	0.13	0.24	0.33	0.43	0.52	0.61	0.69	0.78	0.86	0.94	1.33
198.8	650	0.13	0.23	0.33	0.42	0.51	0.59	0.68	0.76	0.84	0.92	1.30
214.1	700	0.12	0.22	0.32	0.40	0.49	0.57	0.66	0.74	0.81	0.89	1.26
229.4	750	0.12	0.22	0.30	0.39	0.47	0.55	0.63	0.71	0.78	0.86	1.22
244.6	800	0.11	0.21	0.29	0.38	0.45	0.53	0.61	0.68	0.75	0.83	1.17
259.9	850	0.11	0.20	0.28	0.36	0.43	0.51	0.58	0.65	0.72	0.79	1.12
275.2	900	0.10	0.19	0.27	0.34	0.41	0.49	0.55	0.62	0.69	0.75	1.07
290.5	950	0.10	0.18	0.25	0.33	0.39	0.46	0.53	0.59	0.65	0.72	1.01
305.8	1000	0.09	0.17	0.24	0.31	0.37	0.44	0.50	0.56	0.62	0.68	0.96
321.1	1050	0.09	0.16	0.23	0.29	0.35	0.41	0.47	0.53	0.59	0.64	0.91
336.4	1100	0.08	0.15	0.21	0.28	0.33	0.39	0.45	0.50	0.55	0.60	0.86
351.7	1150	0.08	0.14	0.20	0.26	0.31	0.37	0.42	0.47	0.52	0.57	0.81
367.0	1200	0.07	0.13	0.19	0.24	0.29	0.34	0.39	0.44	0.49	0.54	0.76
382.3	1250	0.07	0.13	0.18	0.23	0.28	0.32	0.37	0.41	0.46	0.50	0.71
397.5	1300	0.06	0.12	0.17	0.21	0.26	0.30	0.35	0.39	0.43	0.47	0.67
412.8	1350	0.06	0.11	0.16	0.20	0.24	0.28	0.32	0.36	0.40	0.44	0.62
428.1	1400	0.06	0.10	0.15	0.19	0.23	0.26	0.30	0.34	0.37	0.41	0.58
443.4	1450	0.05	0.10	0.14	0.17	0.21	0.25	0.28	0.32	0.35	0.38	0.54
458.7	1500	0.05	0.09	0.13	0.16	0.20	0.23	0.26	0.29	0.33	0.36	0.51

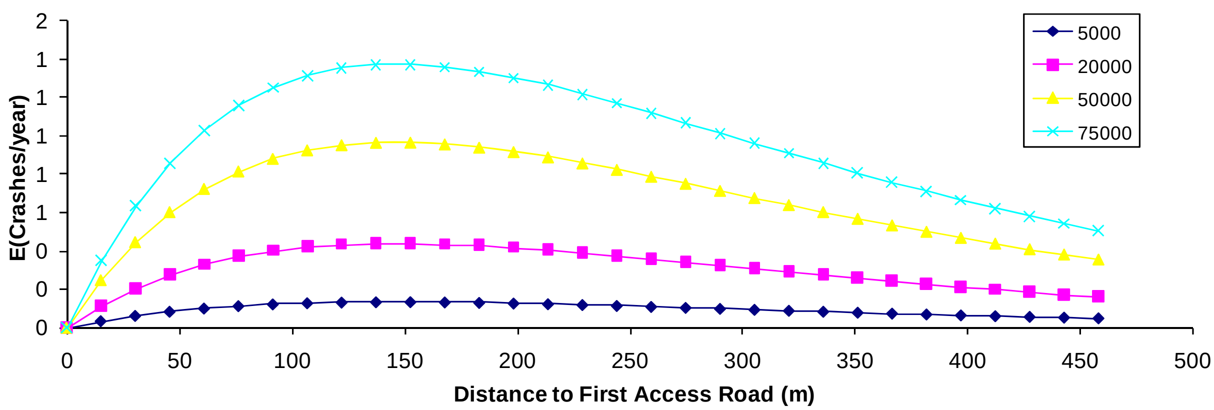


Figure 38. 95% Lower Confidence Limit of the Expected Number of Yearly Crashes As a Function of the Access Section Length and AADT

In an attempt to validate the developed model, the AADT and access road spacing parameters for each of the 186 sites were input to the linear regression and negative binomial GLM models and the expected number of crashes was estimated. A comparison between the observed and estimated crashes revealed a reasonable level of correlation (Pearson correlation coefficient of 0.24) between the observed and linear regression model estimated crash rates, as illustrated in Figure 39. However, a high level of variability is observed in the data. The figure also clearly demonstrates that the negative binomial GLM underestimates the number of crashes significantly.

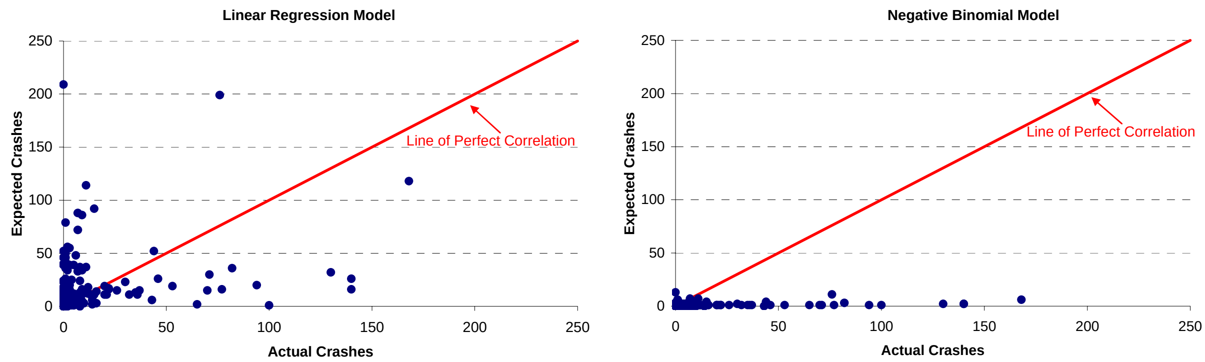


Figure 39. Comparison of Actual and Expected Crashes (All 186 Sites)

A key desire of DOTs is to identify the minimum distance from a freeway ramp to provide access to local businesses. The model developed as part of this research effort was utilized to compute the crash rate associated with alternative section spacing, as summarized in Table 12. The results demonstrate an eight-fold decrease in the crash rate over an access road spacing ranging from 0 ft to 1000 ft. An increase in the minimum spacing from 300 ft to 600 ft results in a 50% reduction in the crash rate.

Table 12. Impact of Access Road Spacing on Annual Crash Rate (AADT = 20,000 veh/day)

Distance to First Access Road					Distance to First Intersection				
L (ft)	L (m)	Crashes per 10^6 VMT	Relative	Relative	L (ft)	L (m)	Crashes per 10^6 VMT	Relative	Relative
0	0.0	10.07	1.00	8.14	0	0.0	10.07	1.00	3.53
50	15.2	9.07	0.90	7.33	50	15.2	9.46	0.94	3.31
100	30.5	8.17	0.81	6.60	100	30.5	8.88	0.88	3.11
150	45.7	7.35	0.73	5.94	150	45.7	8.34	0.83	2.92
200	61.0	6.62	0.66	5.35	200	61.0	7.83	0.78	2.74
250	76.2	5.96	0.59	4.82	250	76.2	7.35	0.73	2.57
300	91.4	5.37	0.53	4.34	300	91.4	6.90	0.69	2.42
350	106.7	4.83	0.48	3.91	350	106.7	6.48	0.64	2.27
400	121.9	4.35	0.43	3.52	400	121.9	6.08	0.60	2.13
450	137.2	3.92	0.39	3.17	450	137.2	5.71	0.57	2.00
500	152.4	3.53	0.35	2.85	500	152.4	5.36	0.53	1.88
550	167.6	3.18	0.32	2.57	550	167.6	5.04	0.50	1.76
600	182.9	2.86	0.28	2.31	600	182.9	4.73	0.47	1.66
650	198.1	2.58	0.26	2.08	650	198.1	4.44	0.44	1.55
700	213.4	2.32	0.23	1.88	700	213.4	4.17	0.41	1.46
750	228.6	2.09	0.21	1.69	750	228.6	3.91	0.39	1.37
800	243.8	1.88	0.19	1.52	800	243.8	3.67	0.36	1.29
850	259.1	1.69	0.17	1.37	850	259.1	3.45	0.34	1.21
900	274.3	1.53	0.15	1.23	900	274.3	3.24	0.32	1.13
950	289.6	1.37	0.14	1.11	950	289.6	3.04	0.30	1.07
1000	304.8	1.24	0.12	1.00	1000	304.8	2.86	0.28	1.00

CONCLUSIONS

- The research effort developed a methodology to quantify the safety impacts of different access road spacing standards. The research demonstrated that a modified Poisson regression model with an over-dispersion parameter (inflates the variability) and a least square LRM approach can be applied to crash data to develop crash prediction models. The proposed LRM approach involves creative manipulation of the data to satisfy the least square LRM assumptions; namely, error structure normality and homoscedasticity. Both models, which were developed using data from 186 access road sections in the state of Virginia, were found to be superior to the state-of-the-art negative binomial maximum likelihood model because they are not influenced (through data aggregation) by the prevalence of the large number of zero observations that are typical of crash data.

- Using these models, the study developed lookup tables that can be used by policy makers to assess the safety implications of different access spacing regulations. It is anticipated that these tables will assist VDOT in developing access management guidelines.

RECOMMENDATIONS

1. *VDOT's Asset Management Division should use the tables developed in this research to establish and implement access spacing requirements.* Further, the Virginia Transportation Research Council should consider sponsoring research to quantify the operational impacts of varying access spacing requirements. Various measures of effectiveness can be considered, including vehicle delay, stops, fuel consumption, and environmental impacts.

2. *The Virginia Transportation Research Council should consider sponsoring research to quantify the operational impacts of access management spacing.* The study would consider different arterial demand levels, demand distributions (left, through, and right volume percentages), access road types (left, right, or through), type of intersection (signalized, stop controlled, or uncontrolled), type of access road (commercial versus residential), availability of accelerations lanes at the off-ramp, etc.

3. *The Virginia Transportation Research Council should consider sponsoring research of site access management strategies (e.g., end islands) that could be added to enhance safety.*

COSTS AND BENEFITS ASSESSMENT

An attempt was made to convert the safety impacts of alternative access road spacing using a weighted average crash cost. The weighted average crash cost was computed considering that 0.6%, 34.8%, and 64.6% of the crashes were fatal, injury, and property damage crashes, respectively. These proportions were generated from the field observed data. The cost of each of these crashes was provided by VDOT as $3,760,000, $48,200, and $6,500 for fatal, injury, and property damage crashes, respectively. This provided an average crash cost of $43,533. This average cost was multiplied by the number of crashes per mile to compute the cost associated with different access spacing scenarios, as summarized in Table 13. It is anticipated that Table 13 can assist policy makers in quantifying the trade-offs of different access management regulations.

Table 13. Cost of Crashes Considering Crashes per Mile (Million Dollars)

L (m)	L (ft)	AADT (veh/day)										
		5000	10000	15000	20000	25000	30000	35000	40000	45000	50000	75000
0.0	0											
15.3	50	0.87	1.58	2.24	2.87	3.47	4.06	4.64	5.21	5.76	6.31	8.94
30.6	100	0.78	1.42	2.02	2.58	3.13	3.66	4.18	4.69	5.19	5.68	8.05
45.9	150	0.71	1.28	1.82	2.32	2.82	3.30	3.76	4.22	4.67	5.11	7.25
61.2	200	0.64	1.15	1.63	2.09	2.54	2.97	3.39	3.80	4.21	4.61	6.53
76.5	250	0.57	1.04	1.47	1.89	2.28	2.67	3.05	3.42	3.79	4.15	5.88
91.7	300	0.52	0.93	1.33	1.70	2.06	2.41	2.75	3.08	3.41	3.73	5.29
107.0	350	0.46	0.84	1.19	1.53	1.85	2.17	2.47	2.78	3.07	3.36	4.77
122.3	400	0.42	0.76	1.07	1.38	1.67	1.95	2.23	2.50	2.77	3.03	4.29
137.6	450	0.38	0.68	0.97	1.24	1.50	1.76	2.01	2.25	2.49	2.73	3.86
152.9	500	0.34	0.61	0.87	1.12	1.35	1.58	1.81	2.03	2.24	2.46	3.48
168.2	550	0.30	0.55	0.78	1.00	1.22	1.42	1.63	1.82	2.02	2.21	3.13
183.5	600	0.27	0.50	0.71	0.90	1.10	1.28	1.46	1.64	1.82	1.99	2.82
198.8	650	0.25	0.45	0.64	0.81	0.99	1.16	1.32	1.48	1.64	1.79	2.54
214.1	700	0.22	0.40	0.57	0.73	0.89	1.04	1.19	1.33	1.47	1.61	2.29
229.4	750	0.20	0.36	0.52	0.66	0.80	0.94	1.07	1.20	1.33	1.45	2.06
244.6	800	0.18	0.33	0.46	0.59	0.72	0.84	0.96	1.08	1.20	1.31	1.86
259.9	850	0.16	0.30	0.42	0.54	0.65	0.76	0.87	0.97	1.08	1.18	1.67
275.2	900	0.15	0.27	0.38	0.48	0.58	0.68	0.78	0.88	0.97	1.06	1.50
290.5	950	0.13	0.24	0.34	0.43	0.53	0.62	0.70	0.79	0.87	0.96	1.35
305.8	1000	0.12	0.22	0.31	0.39	0.47	0.55	0.63	0.71	0.79	0.86	1.22
321.1	1050	0.11	0.19	0.28	0.35	0.43	0.50	0.57	0.64	0.71	0.77	1.10
336.4	1100	0.10	0.17	0.25	0.32	0.38	0.45	0.51	0.58	0.64	0.70	0.99
351.7	1150	0.09	0.16	0.22	0.29	0.35	0.40	0.46	0.52	0.57	0.63	0.89
367.0	1200	0.08	0.14	0.20	0.26	0.31	0.36	0.42	0.47	0.52	0.57	0.80
382.3	1250	0.07	0.13	0.18	0.23	0.28	0.33	0.37	0.42	0.47	0.51	0.72
397.5	1300	0.06	0.11	0.16	0.21	0.25	0.30	0.34	0.38	0.42	0.46	0.65
412.8	1350	0.06	0.10	0.15	0.19	0.23	0.27	0.30	0.34	0.38	0.41	0.59
428.1	1400	0.05	0.09	0.13	0.17	0.20	0.24	0.27	0.31	0.34	0.37	0.53
443.4	1450	0.05	0.08	0.12	0.15	0.18	0.22	0.25	0.28	0.31	0.33	0.47
458.7	1500	0.04	0.08	0.11	0.14	0.17	0.19	0.22	0.25	0.28	0.30	0.43

ACKNOWLEDGMENTS

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APPENDIX A

VDOT STANDARDS

VDOT

FHWA

ACCESS CONTROL AGREEMENT FOR INTERSTATE INTERCHANGES IN VIRIGINIA BETWEEN THE VIRGINIA DEPARTMENT OF TRANSPORTATION AND THE FEDERAL HIGHWAY ADMINISTRATION

Purpose

The purpose of this agreement is to clarify the design standards and implementation policy for access control at new and modified interchanges along Interstates in Virginia.

Policy

One of the key components to determining the design standard for control of access is the forecast population of the area of the interchange. The length of access control that is required on the cross roads of an interchange is dependent upon the planning designation (Urban, Planned Urban, or Rural) of the area in which the interchange is located. The "Urban" designation is given to those census-determined areas that have a population of 5,000 or more. The "Planned Urban" designation is given to those areas that have a population forecast of 5,000, or more, in an approved Long Range Plan. A "Rural" area is any area not Urban or Planned Urban.

All new interchanges, as well as existing interchanges with proposed improvements, must be evaluated for safety and operational needs to determine the appropriate lengths of access restrictions. VDOT should make every effort to control access on the cross roads for the distance that is recommended in the study. The minimum length of Limited Access Rights shall not be less than the following:

Urban Areas – The AASHTO standard for length of Control of Access should be as much as practical but shall be a minimum of 100 ft. beyond the proposed ramp terminal.

Planned Urban Areas – The AASHTO standard for length of Control of Access is a minimum of 100 ft beyond the proposed ramp terminal. In a situation where an access point exists less than 300 ft. and over 100 ft. from the ramp terminal, VDOT should make a concerted effort to relocate the access outside the 300 ft. or as far from the proposed ramp terminal as possible. In a situation where an access point does not already exist, VDOT should acquire a minimum of 300 ft. of Controlled Access beyond the ramp terminal.

Rural Areas – The AASHTO standard length for Control of Access is a minimum of 300 ft. beyond the ramp terminal.

All access requests not meeting the minimum lengths listed above would require a design exception approved by FHWA.

Field Operations Team Leader
Federal Highway Administration

4/28/04
Date

State Location and Design Engineer
Virginia Department of Transportation

4/28/04
Date

BATES

COMMONWEALTH *of* VIRGINIA

DEPARTMENT OF TRANSPORTATION
1401 EAST BROAD STREET
RICHMOND, 23219

J.T. MILLS
DIVISION ADMINISTRATOR

RECEIVED

September 21, 1999

MEMORANDUM

To : District Administrators
District Construction Engineers
Resident Engineers
District L&D Engineers

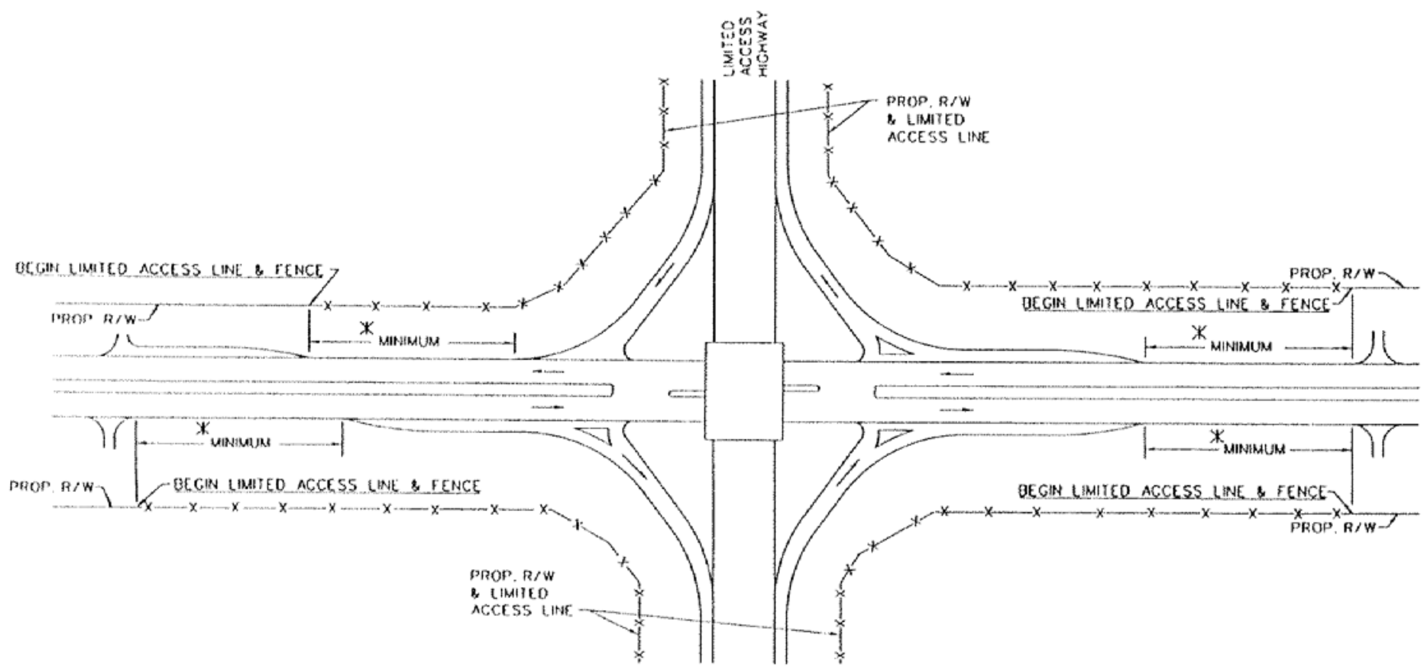
SUBJECT: Application of Limited Access Boundaries

There has recently been much discussion over the application of limited access boundaries on roads intersecting with Limited Access Highways. The Department has followed the guidance provided by AASHTO for some time – that is, to extend the limited access a minimum of 300' on rural interchanges and a minimum of 100' on urban interchanges beyond the end of the interchange ramps. The Department's position has been communicated by the Commissioner and Chief Engineer, and details concerning the application of these limits are contained in our Road Design Manual. None the less, there remains some confusion when situations arise that require the application of these limits.

The attached sketch is provided in an effort to clarify the application of these limits.

J. T. Mills
State Location & Design Engineer

cc: Mr J.G. Browder, Jr.
Attachment



* 100' URBAN/300' RURAL, MEASURED FROM THE ACCELERATION/DECELERATION LANE TAPER TO THE CLOSEST POINT (TURN LANE TAPER/ENTRANCE FLARE) OF ENTRANCE.

DEPICTING LIMITED ACCESS RIGHT OF WAY

APPENDIX B

ADDITIONAL EXPLORATORY ANALYSIS

The research team recomputed all the values when the distance to the first access is considered from the end of the taper of the ramp.

Exploratory Data Analysis

In this section we present a qualitative data analysis conducted to identify the impact of geometric and traffic factors on the crash rates as a function of the distance from the off-ramp to the first access road. The crash rate is computed as

$$CR = \frac{C}{(AADT \times 365 \times 5) \times 10^{-6} \times L_2} \quad [B\text{-}1]$$

where CR is the crash rate in crashes per million-veh-km; C is the number of crashes along a section, L_2 is the length of the section which is the distance between the freeway off-ramp and the first intersection (km).

Crash rates were computed for each of the different categories. For each category the 300, 660, 750, 990 and 1320 ft, (90, 200, 230 and 400 m) lines are also plotted to reflect the current Virginia Standards and more conservative ones (7). When the distance to the first access is considered from the end of the taper, the following results are obtained (Table B-1).

Table B-1. Average Crash Frequency, Rate, Distances and AADT for the Different Categories

Category	Sub category	# sections	%	Crash F[1]	CR[2]	CR d1[3]	#Sec	CR d2[4]	# Sec	L1(mile)[5]	L2(mile)[6]	AADT
Total	Total	186	100	12.24	5.94	9.82	70	3.99	108	0.11	0.19	19456
Area	Urban	107	58	17.61	7.119	5.36	43	3.18	36	0.19	0.11	28299
	Rural	79	42	4.97	13.36	8.33	34	4.56	35	0.10	0.18	7479
Median	Yes	126	68	15.33	4.29	7.94	40	3.20	53	0.12	0.17	25882
	No	60	32	5.75	9.91	13.34	35	5.90	18	0.08	0.33	5962
Type of Control First access	Signalized	67	36	17.61	3.71	6.27	23	3.04	27	0.13	0.20	30063
	Unsignalized	119	64	9.22	7.19	11.50	54	4.50	44	0.09	0.18	13485
LTL	Yes	78	42	18.45	3.84	6.07	19	3.63	34	0.13	0.19	28581
	No	108	58	7.76	7.45	11.39	55	4.32	36	0.09	0.18	12866
Acceleration Lane	Yes	76	41	18.54	4.37	8.20	25	3.64	30	0.13	0.19	31399
	No	110	59	7.89	7.03	10.58	53	4.25	40	0.09	0.18	11205
Number of lanes	2	57	31	4.98	8.639	11.37	36	4.94	16	0.08	0.22	5204
	4	95	51	13.59	5.596	10.09	33	3.98	42	0.12	0.16	17744
	More than 6	34	18	19.57	2.37	2.60	9	2.43	20	0.12	0.20	48133
Type of First Access	Intersection	62	33	17.69	3.63	6.39	10	4.27	31	0.16	0.16	29861
	Right	89	48	9.80	6.57	8.92	51	4.25	27	0.07	0.18	16065
	Left	35	19	8.80	8.44	15.37	16	2.54	13	0.09	0.23	9648

(1) Average Crash Frequency for 5 years' exposure
(2) Average Annual Crash Rate Crashes per million-veh-miles
(3) CR d1 Average Annual Crash Rate for sections with distance from the off-ramp to the first access between 0-300 feet
(4) CR d2 Average Annual Crash Rate for sections with distance from the off-ramp to the first access between 300-750 feet
(5) L1 Distance from the off-ramp to first access (miles)
(6) L2 Distance from the off-ramp to first intersection (miles)

Rural vs. Urban

The rural sections have a higher average crash rate when compared to urban sections with values of 13.36 and 7.119 crashes per million vehicle miles of travel, respectively. These differences are statistically significant (p value of 0.043). For sections shorter than d1, the crash rate is 8.33 for rural roads and 5.36 for urban roads. In the case of distance d2 there is a significant reduction (>40%) in the crash rates (Figure B-1).

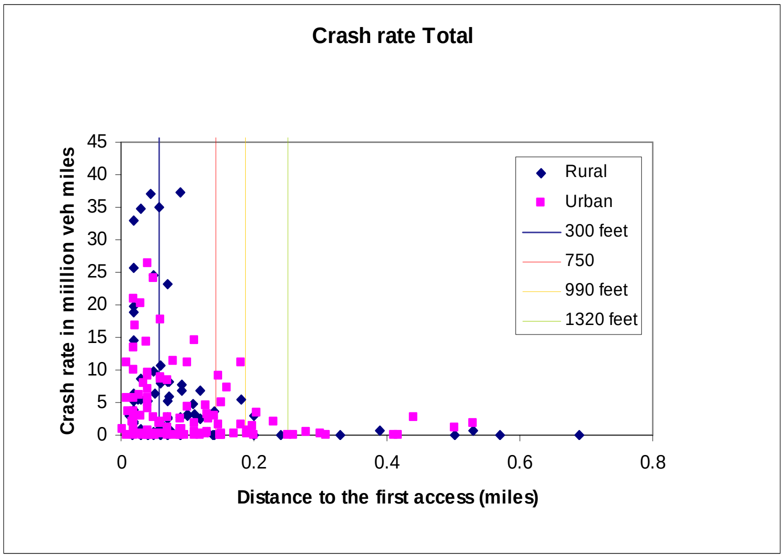


Figure B-1. Crash Rate for the Urban/Rural Category

Median

The crash rate without medians is 9.91 compared with 4.29 for sections with medians. The differences of the means are statistically significant (p value of 0.0066). For sections shorter than d1, the crash rate is 13.34 for sections with no median, and 7.94 with a median. For access distances d2, the numbers are reduced to 5.90 and 3.2, respectively (Figure B-2).

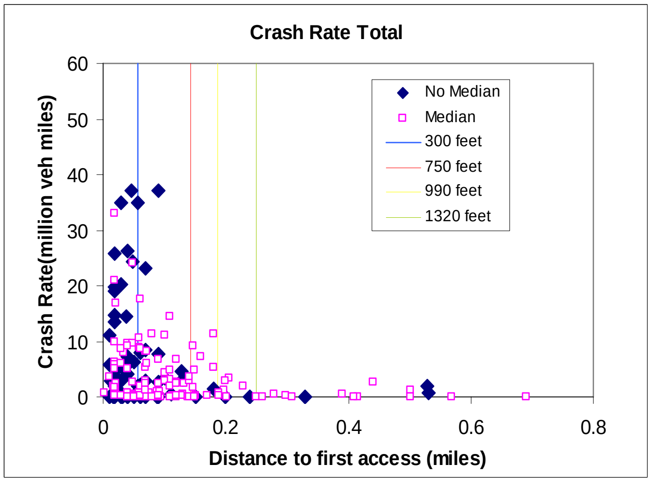


Figure B-2 Crash Rate for the Median/No Median Category

Type of First Access

The average crash rate for all of the accidents was found to be lower when the first access was an intersection (3.63) and higher for right and left access roads (6.57 and 8.44, respectively). The average crash rates for distances d2 experienced a significant reduction compared with the ones with distances d1 (less than 300 ft) with more than 40% reduction for all three cases (Figure B-3).

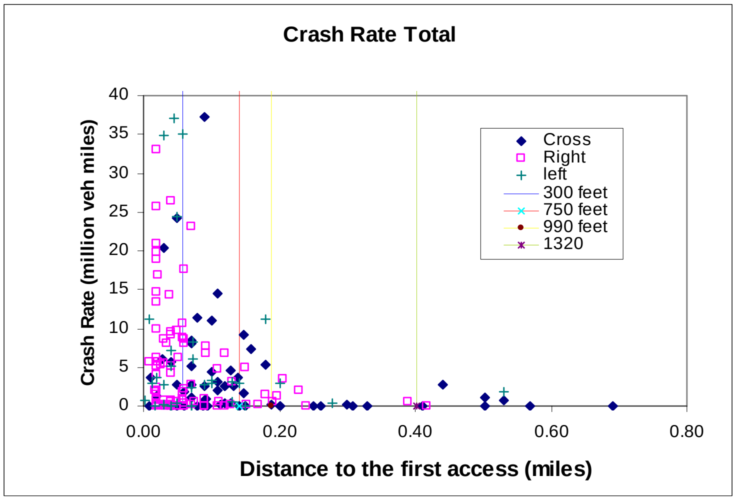


Figure B-3. Distribution of Distances and Crash Rate for the Type of Access Category

Number of Lanes

The crash rate for the system decreases from 8.639 for two lanes to 5.596 for four lanes and 2.31 for 6 lanes, respectively. The average rates for sections with L1 less than d1 are 11.37 for two lanes, 10.09 for four lanes, and 2.60 for sections with six or more lanes. For the sections where this distance is increased to d2, there is a decrease in the crash rate to 4.94, 3.98, and 2.43, respectively (Figure B-4).

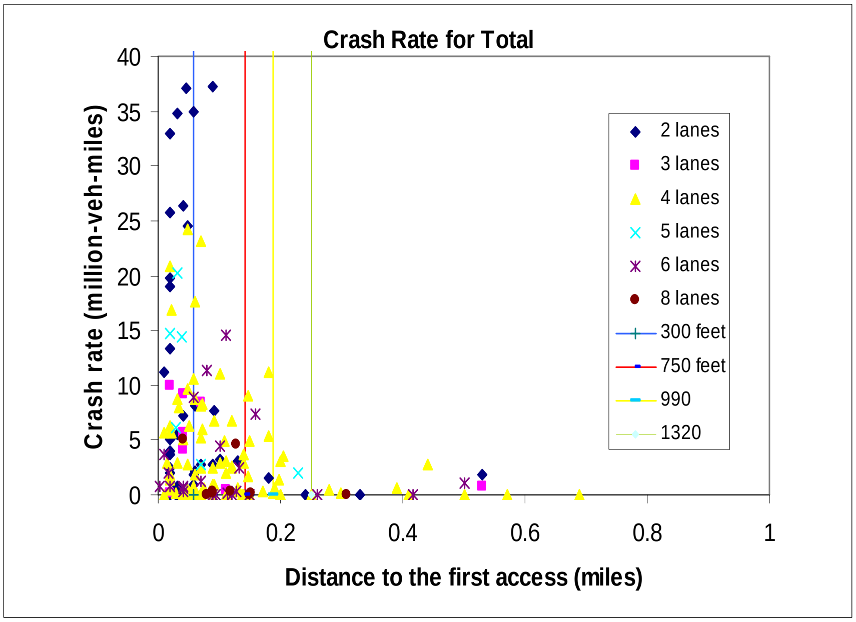


Figure B-4. Crash Rate for the Number of Lanes Category

Left Storage Lane

The average crash rate is 7.45 for sections without a left turn lane compared with 3.84 for sections with a left turn lane. The differences are statistically significant (p value of 0.017). Sections without left turn lanes with distance to the first access shorter than d1 experience an average crash rate of 11.39. This crash rate is reduced by 62% for sections with distance to the first access d2. In the case of sections with a left turn lane, the reduction is 40% (Figure B-5).

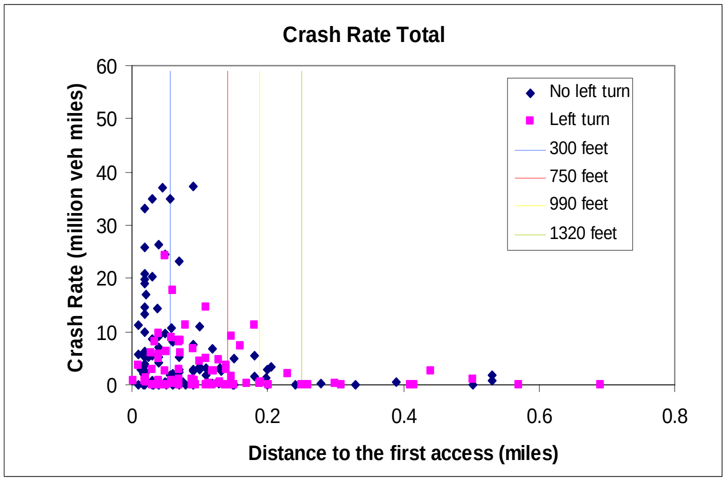


Figure B-5. Crash Rate as a Function of the Presence of a Left Turn Storage Lane

Type of Control at First Intersection

The average crash rate was 7.19 for unsignalized sections and 3.71 for signalized sections. The crash rate with L1 less than d1 was 11.50 for unsignalized sections and 6.27 for signalized intersections. For unsignalized sections with access distances between 300 ft and 750 ft, the rate was reduced by 60.8% in comparison to the sections with L1 less than d1. A similar situation is experienced for signalized sections with a drop in rate of 51.5% (Figure B-6).

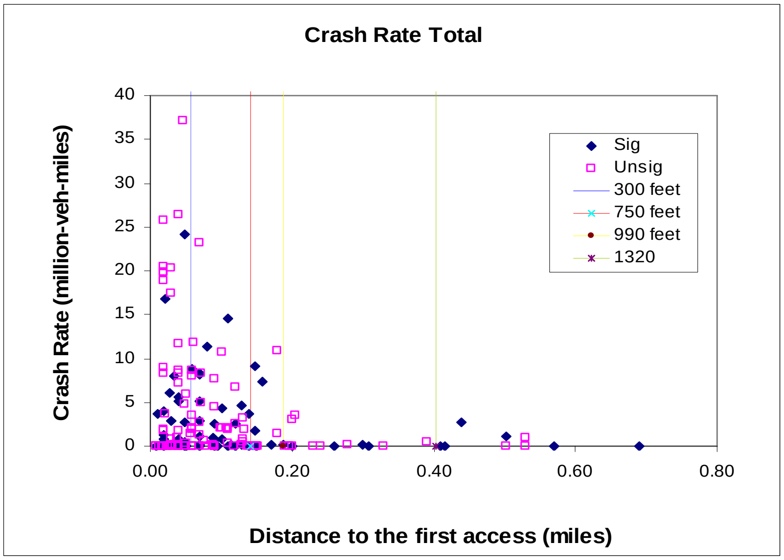


Figure B-6. Crash Rate for the Signalized/Unsignalized Category

Acceleration Lane

Sections with no acceleration lane have a higher crash rate of 7.03 compared with the sections that have an acceleration lane 4.37. The crash rate for sections having no acceleration lanes and distances to the first access more than 300 ft but less than 750ft exceed the crash rate of sections having acceleration lanes by 14%; however, for access distance d1, sections with acceleration lanes experienced bigger reductions than the ones that do not have an acceleration lane (Figure B-7).

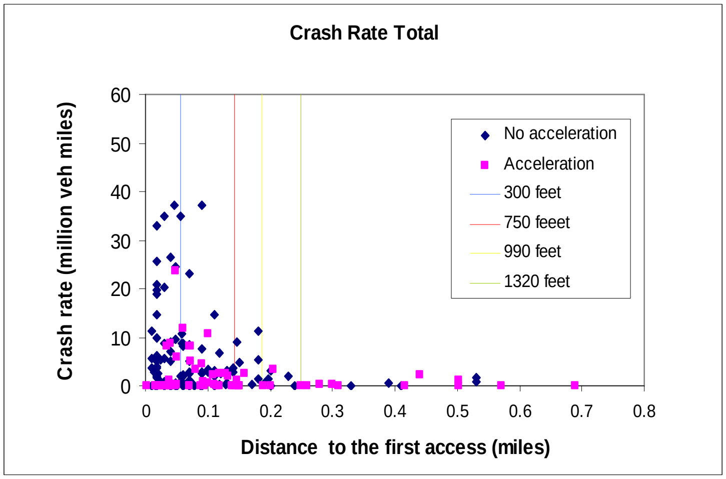


Figure B-7. Crash Rate as a Function of the Presence of Acceleration Lane

APPENDIX C

ADDITIONAL RESULTS

This section describes the development of crash models considering the distance to the first access as the distance from the end of the taper of the off-ramp to the centerline of the first access road.

Poisson or Negative Binomial Model Approach

When the distance is considered from the taper of the ramp, the following results are obtained.

The estimated model is shown in Equation 1 and the parameters for both models with and without zeros are shown in Table C-1.

$$\frac{C}{L_2} = \left(\frac{(1825)^{0.7228}}{10^6}\right) \times \exp\left(6.5430 - 65.5357 \times L_1 + 0.7228 \times \ln(V)\right) \qquad \text{[C-1]}$$

Table C- 1. Summary Results of Poisson Regression

a. Analysis of Parameter Estimates						
	With Zeros			**Without Zeros**		
Parameter	DF	Estimate	Pr > Chi2	DF	Estimate	Pr > Chi2
Intercept	1	6.5430	<.0001	1	7.5064	<.0001
L_1	1	-5.5357	<.0001	1	-4.7855	<.0001
Ln(V)	0	0.7228		0	0.6696	
Scale	0	1.0000		0	1.0000	
b. Criteria For Assessing Goodness of Fit						
	With Zeros			**Without Zeros**		
Criterion	DF	Value	Value/DF	DF	Value	Value/DF
Deviance	184	103987.4234	565.1490	145	180229.4534	1242.9617
Scaled Deviance	184	103987.4234	565.1490	145	180229.4534	1242.9617
Pearson Chi-Square	184	145260.5458	789.4595	145	249265.4475	1719.0721
Scaled Pearson X2	184	145260.5458	789.4595	145	249265.4475	1719.0721
Log Likelihood		335542.7320			929591.6906	

Assessing the goodness of fit for the model, one can observe that the Poisson regression model suffers from over-dispersion (565.1490>1 for data that include the zero and 1242.9617>1 for data without the zeros). The Negative Binomial model was fit to the data in Equation 4 and the model parameters are summarized in Table C-2. All the parameters are found to be significant (p values <0.0001), as shown in Table C-2.

Table C-2. Summary Results for Negative Binomial Regression

a. Analysis of Parameter Estimates						
	With Zeros			**Without Zeros**		
Parameter	DF	Estimate	Pr > Chi2	DF	Estimate	Pr > Chi2
Intercept	1	6.3781	<.0001	1	7.1578	<.0001
L_1	1	-3.703	<.0001	1	-3.2221	<.0001
Ln(V)	0	0.7555		0	0.6820	
Dispersion	1	3.3807		1	1.3152	
b. Criteria For Assessing Goodness of Fit						
	With Zeros			**Without Zeros**		
Criterion	DF	Value	Value/DF	DF	Value	Value/DF
Deviance	184	225.6291	1.2262	145	175.3168	1.2091
Scaled Deviance	184	225.6291	1.2262	145	175.3168	1.2091
Pearson Chi-Square	184	126.0044	0.6848	145	182.7911	1.2606
Scaled Pearson X2	184	126.0044	0.6848	145	182.7911	1.2606
Log Likelihood		386916.1838			840538.36	

Assessing the goodness of fit for the model, deviance divided by the degrees of freedom is quite small (1.2063 and 1.2011) as shown in the table, showing a better model than the Poisson model.

The second approach is the Modified Poisson regression by adding an over-dispersion parameter to the model to inflate the variability of the estimators. The model was fit to the data and the parameters are summarized in Table C-3. All the parameters are found to be significant (p values of <0.0001), as shown in Table C-3.

Table C- 3. Modified Poisson Regression Results

a. Analysis of Parameter Estimates						
	With Zeros			**Without Zeros**		
Parameter	DF	Estimate	Pr > Chi2	DF	Estimate	Pr > Chi2
Intercept	1	6.5430	<.0001	1	7.5064	<.0001
L_1	1	-5.5357	<.0001	1	-4.7855	<.0001
Ln(V)	0	0.7228		0	0.6696	
Scale	0	23.7729		0	35.2557	
b. Criteria For Assessing Goodness of Fit						
	With Zeros			**Without Zeros**		
Criterion	DF	Value	Value/DF	DF	Value	Value/DF
Deviance	184	103987.4234	565.1490	145	180229.4534	1242.9617
Scaled Deviance	184	184.0000	1.0000	145	145.0000	1.0000
Pearson Chi-Square	184	145260.5458	789.4595	145	249265.4475	1719.0721
Scaled Pearson X2	184	257.0305	1.3969	145	200.5415	1.3830
Log Likelihood		593.7243			747.8844	

The results of the test for the coincidence of the two regression models test are presented in Table C-4. It shows, considering a significance level (α =0.05), that none of the variables are statistically significant.

Table C-4. Summary of Significance of Other Variables

Condition		B_0	B_1	P-value
Median?	Yes	6.4091	-4.595	0.414
	No	6.7896	-8.7878	
Acceleration. Lane?	Yes	6.7844	-6.5321	0.430
	No	6.4231	-5.0816	
Urban?	Yes	6.4455	-4.5932	0.487
	No	6.7383	-7.8982	
Left Turn Storage?	Yes	6.4482	-4.3805	0.530
	No	6.6336	-7.0602	
Number of Legs	1	6.6139	-6.3848	0.356
	>1	6.1787	-2.0907	
Signalized ?	Yes	6.284	-4.2014	0.408
	No	6.6865	-6.4157	
Number of Lanes	2	6.562	-5.7194	0.958
	>2	6.4591	-4.8952	
Type of First Access	Yes	6.68	-5.2351	0.105
	No	6.6038	-9.5212	

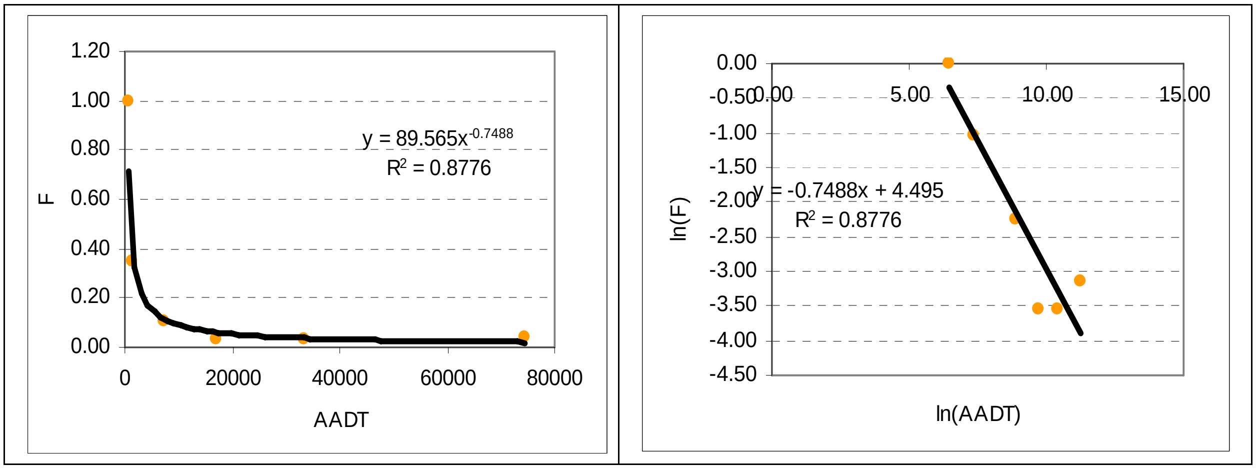
The model can be cast as

$$D = \begin{cases} 1, & CR = \exp\left((b_0 + b_2) + (b_1 + b_3)L_1\right) \\ 0, & CR = \exp\left(b_0 + b_1 L_1\right) \end{cases}. \qquad \text{[C-2]}$$

Linear Regression Modeling Approach

A least squares LRM was then fit to the log-transformed data producing an R^2 of 0.877, as illustrated in Figure C-1. The model was statistically significant and both the intercept and slope coefficients were significant (p = 0.02 and 0.005, respectively). Consequently, the exponent of the AADT for utilization in the exposure measure is 0.7488, which is very similar to what was derived from the negative binomial fit to the data (p = 0.749).

Once the exponent of the AADT was estimated, the crash rate was computed for each of the 186 study sections. A linear regression model was fit to the data. As demonstrated in Figure C-2 there was insufficient evidence to reject the data error normality and homoscedasticity assumption for the log-transformed data (p=0.348) and thus a least squares GLM could be applied to the data.

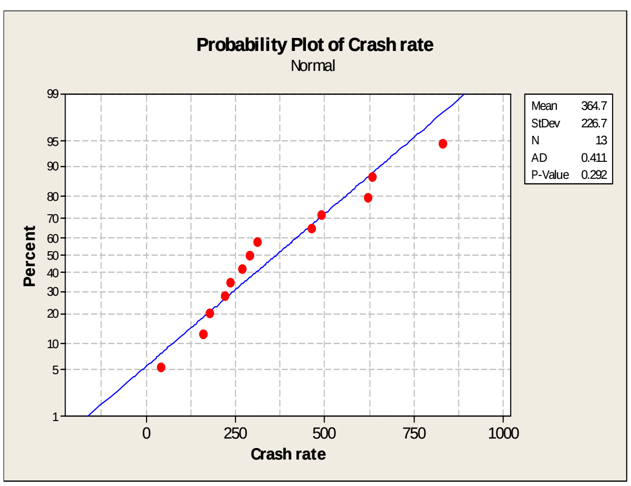


Regression Statistics	
Multiple R	0.93680677
R Square	0.87760692
Adj R square	0.84700864
SE	0.57071845
Observations	6

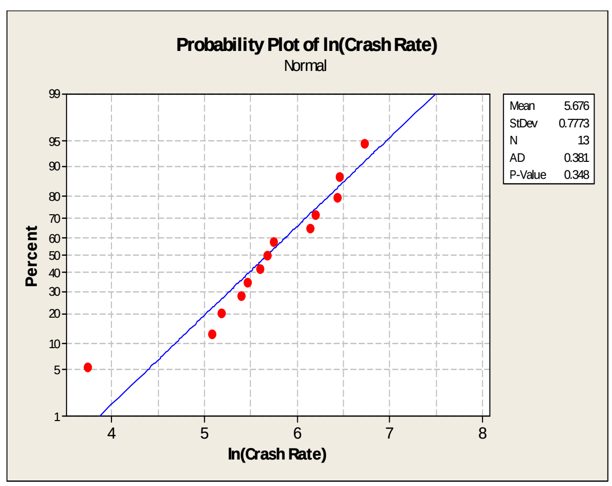
ANOVA

	df	*SS*	*MS*	*F*	*Sig. F*	
Regression	1	9.342153	9.342153	28.68159	0.005864	
Residual	4	1.302878	0.32572			
Total	5	10.64503				
	Coeffs.	*SE*	*t Stat*	*P-value*	*Low95%*	*Up 95%*
Intercept	4.49496577	1.281651	3.507169	0.024735	0.936526	8.053406
X Variable 1	0.74879519	0.139817	-5.35552	0.005864	-1.13699	-0.3606

Figure C-1. Computation of Exposure Measures



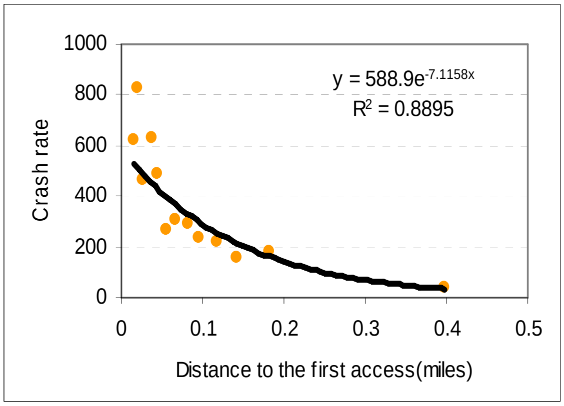
(a) Normality Test on Crash Rate

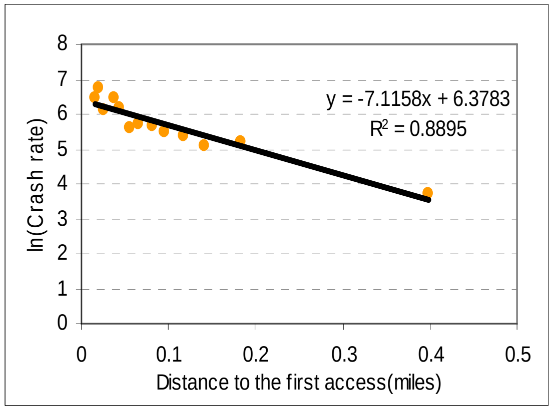


(b) Normality Test on Log-transformed Crash Rate

Figure C-2. Test of Normality for Crash Rate Data

A robust linear regression was applied to the data to derive the model parameters and remove outlier data. The results of the analysis demonstrate a statistically significant model (F=88.56 and p<0.0005) with an R^2 of 0.88. The intercept and L_1 coefficients are statistically significant (p<0.0005 and p<0.0005, respectively) with values of 6.3783 and -7.1158, respectively (Figure C-3).





Regression Statistics	
Multiple R	0.943144
R Square	0.88952
Adj R square	0.879476
SE	0.269844
Observations	13

ANOVA

	df	*SS*	*MS*	*F*	*Sig. F*
Regression	1	6.44897	6.44897	88.56557	1.35E-06
Residual	11	0.800973	0.072816		
Total	12	7.249943			

	Coeff.	*SE*	*t Stat*	*P-value*	*Low95%*	*Up95%*
Intercept	6.378259	0.105716	60.33378	3.21E-15	6.145579	6.610939
L1	-7.1158	0.756121	-9.41093	1.35E-06	-8.78001	-5.45159

Figure C-3. Crash Prediction Model Considering Nearest Access Point

Similarly, a regression model was fit to the data considering the independent variable as the distance to the first intersection. A similar robust regression was applied to the data to derive the model intercept and slope. Given that the intercept confidence limits included the value of intercept of the first model, the intercept was kept constant in both models. A regression was then performed to estimate the optimum slope. The model is significant (p<0.0005) with an R^2 of 0.784. The slope of the line is significant (p<0.0005) with a value of -4.5391 (Figure C-4).

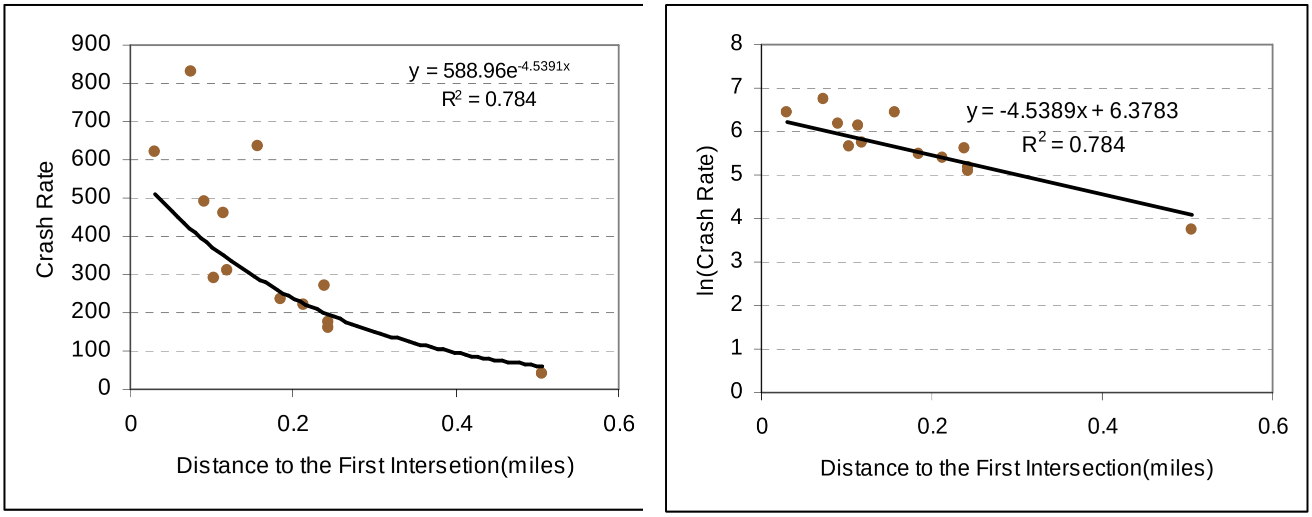


Figure C-4. Crash Prediction Model Considering Nearest Intersection

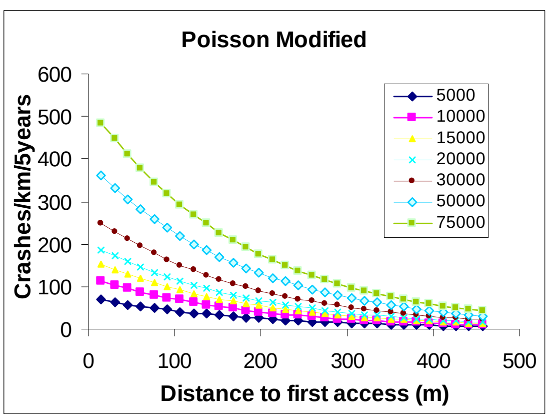
The crash rate in vehicle-miles traveled (VMT) considering an exponent of 1.0 (*CR"*) is computed as

$$CR'' = \exp\left(b_0 + (p - 1)\ln 5 + b_1 L_1\right) \times 1.6\left(365V\right)^{p-1}, \text{ or}$$

$$CR'' = \exp\left(b_0 + \ln 1.6 + (p - 1)\ln 5 + (p - 1)\ln 365 + b_1 L_1 + (p - 1)\ln V\right)$$

In an attempt to validate the model, the AADT and access road spacing parameters for the 186 study sections were input into the Modified Poisson Regression model. A comparison between the observed and estimated crashes revealed a reasonable level of correlation (Pearson correlation coefficient 0.25), as illustrated Figure C-5.

L1(feet)	L1(km)	AADT (veh/day) 5000	10000	15000	20000	25000	30000	35000	40000	45000	50000	75000
50	0.01524	68.52	113.08	151.59	186.63	219.29	250.18	279.67	308.01	335.38	361.91	485.16
100	0.03048	62.97	103.93	139.33	171.53	201.55	229.94	257.04	283.09	308.24	332.63	445.91
150	0.04572	57.88	95.52	128.05	157.65	185.24	211.34	236.25	260.18	283.31	305.72	409.83
200	0.06096	53.20	87.80	117.69	144.90	170.26	194.24	217.13	239.14	260.39	280.99	376.68
250	0.0762	48.89	80.69	108.17	133.17	156.48	178.52	199.57	219.79	239.32	258.26	346.20
300	0.09144	44.94	74.16	99.42	122.40	143.82	164.08	183.42	202.01	219.96	237.36	318.19
350	0.10668	41.30	68.16	91.38	112.50	132.19	150.81	168.58	185.66	202.16	218.16	292.45
400	0.12192	37.96	62.65	83.98	103.40	121.49	138.61	154.94	170.64	185.81	200.51	268.79
450	0.13716	34.89	57.58	77.19	95.03	111.66	127.39	142.41	156.84	170.77	184.29	247.04
500	0.1524	32.07	52.92	70.94	87.34	102.63	117.09	130.89	144.15	156.96	169.38	227.06
550	0.16764	29.47	48.64	65.21	80.28	94.33	107.61	120.30	132.49	144.26	155.67	208.69
600	0.18288	27.09	44.71	59.93	73.78	86.70	98.91	110.56	121.77	132.59	143.08	191.80
650	0.19812	24.90	41.09	55.08	67.81	79.68	90.91	101.62	111.92	121.86	131.50	176.29
700	0.21336	22.88	37.76	50.62	62.33	73.23	83.55	93.40	102.86	112.00	120.87	162.02
750	0.2286	21.03	34.71	46.53	57.28	67.31	76.79	85.84	94.54	102.94	111.09	148.92
800	0.24384	19.33	31.90	42.76	52.65	61.86	70.58	78.90	86.89	94.61	102.10	136.87
850	0.25908	17.77	29.32	39.31	48.39	56.86	64.87	72.51	79.86	86.96	93.84	125.80
900	0.27432	16.33	26.95	36.13	44.47	52.26	59.62	66.65	73.40	79.92	86.25	115.62
950	0.28956	15.01	24.77	33.20	40.88	48.03	54.80	61.26	67.46	73.46	79.27	106.26
1000	0.3048	13.79	22.76	30.52	37.57	44.15	50.36	56.30	62.00	67.51	72.86	97.67
1050	0.32004	12.68	20.92	28.05	34.53	40.57	46.29	51.74	56.99	62.05	66.96	89.77
1100	0.33528	11.65	19.23	25.78	31.74	37.29	42.54	47.56	52.38	57.03	61.54	82.50
1150	0.35052	10.71	17.67	23.69	29.17	34.27	39.10	43.71	48.14	52.42	56.57	75.83
1200	0.36576	9.84	16.24	21.78	26.81	31.50	35.94	40.17	44.25	48.18	51.99	69.69
1250	0.381	9.05	14.93	20.01	24.64	28.95	33.03	36.92	40.67	44.28	47.78	64.05
1300	0.39624	8.31	13.72	18.39	22.65	26.61	30.36	33.94	37.38	40.70	43.92	58.87
1350	0.41148	7.64	12.61	16.91	20.81	24.46	27.90	31.19	34.35	37.40	40.36	54.11
1400	0.42672	7.02	11.59	15.54	19.13	22.48	25.65	28.67	31.57	34.38	37.10	49.73
1450	0.44196	6.46	10.65	14.28	17.58	20.66	23.57	26.35	29.02	31.60	34.10	45.71
1500	0.4572	5.93	9.79	13.13	16.16	18.99	21.66	24.22	26.67	29.04	31.34	42.01



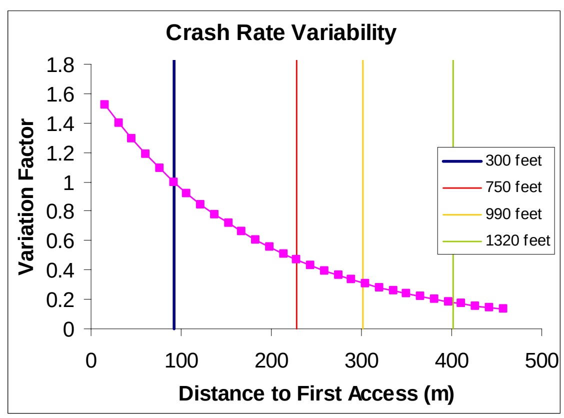


Figure C-5. Variation in the Expected Number of Crashes as a Function to the Distance to the First Access and Comparison of Actual and Expected Crashes