Project No. 15-75

Acceleration, Deceleration and Stopping Sight Distance Criteria for Geometric Design of Highways and Streets

Appendices to the Final Report

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Appendix A Detailed Data Results and Analyses

A.1 Evaluation of Characteristics of Vehicle Fleet

This section provides summary information from headlight height, taillight height, and driver eye height measurements that were conducted in Michigan, North Carolina, and Pennsylvania as shown in Table 4-1, Table 4-2, and Table 4-3, respectively. These tables include separate measurements for passenger cars (e.g., compact sedan, hatchback sedan, etc.) and multipurpose vehicles (i.e., sport utility vehicles, vans, pickup trucks).

Table 4-1. Vehicle Fleet and Driver Eye Height Summary – Michigan Data.

	Headlight Height		Taillight Height		Length from Bumper to Front of Driver Seat	
	Passenger Cars	Multipurpose Vehicles	Passenger Cars	Multipurpose Vehicles	Passenger Cars	Multipurpose Vehicles
Sample Size	495	819	495	819	495	819
Mean (ft)	2.32	2.97	2.99	3.57	8.27	8.29
Std. Deviation (ft)	0.17	0.27	0.23	0.28	0.37	0.38
5th Percentile (ft)	2.10	2.60	2.70	3.15	7.65	7.70
10th Percentile (ft)	2.15	2.70	2.75	3.20	7.85	7.85
15th Percentile (ft)	2.20	2.70	2.80	3.30	7.90	7.92
50th Percentile (ft)	2.30	2.95	3.00	3.60	8.30	8.25

	Top of Headrest Height		Bottom of Headrest Height		Top of Seat Height	
	Passenger Cars	Multipurpose Vehicles	Passenger Cars	Multipurpose Vehicles	Passenger Cars	Multipurpose Vehicles
Sample Size	495	819	495	819	495	819
Mean (ft)	4.16	4.92	3.54	4.28	3.45	4.18
Std. Deviation (ft)	0.20	0.32	0.19	0.33	0.18	0.33
5th Percentile (ft)	3.85	4.50	3.30	3.85	3.20	3.75
10th Percentile (ft)	3.95	4.55	3.35	3.90	3.25	3.85
15th Percentile (ft)	4.00	4.60	3.40	3.95	3.30	3.90
50th Percentile (ft)	4.15	4.85	3.50	4.20	3.45	4.10

Center of Headrest/Driver Eye Height (Average of top and bottom of headrest)		
	Passenger Cars	Multipurpose Vehicles
Sample Size	495	819
Mean (ft)	3.85	4.60
Std. Deviation (ft)	0.20	0.33
5th Percentile (ft)	3.58	4.18
10th Percentile (ft)	3.65	4.23
15th Percentile (ft)	3.70	4.28
50th Percentile (ft)	3.83	4.53

Driver Eye Height Comparison Between Michigan and NCHRP 400 (Fambro et al., 1997)				
	Passenger Cars		Multipurpose Vehicles	
	Michigan	NCHRP-400	Michigan	NCHRP-400
Sample Size	495	875	819	629
Mean (ft)	3.85	3.77	4.60	4.86
Std. Deviation (ft)	0.20	0.18	0.33	0.43
5th Percentile (ft)	3.58	3.48	4.18	4.15
10th Percentile (ft)	3.65	3.55	4.23	4.28
15th Percentile (ft)	3.70	3.59	4.28	4.37
50th Percentile (ft)	3.83	–	4.53	–

Note: – parameters are not provided.

Table 4-2. Vehicle Fleet and Driver Eye Height Summary – North Carolina Data.

	Headlight Height		Taillight Height		Length from Bumper to Front of Driver Seat	
	Passenger Cars	Multipurpose Vehicles	Passenger Cars	Multipurpose Vehicles	Passenger Cars	Multipurpose Vehicles
Sample Size	368	328	368	328	368	328
Mean (ft)	2.31	2.98	2.95	3.58	8.31	8.27
Std. Deviation (ft)	0.18	0.28	0.20	0.29	0.43	0.37
5th Percentile (ft)	2.05	2.60	2.70	3.13	7.57	7.65
10th Percentile (ft)	2.14	2.65	2.75	3.25	7.80	7.75
15th Percentile (ft)	2.15	2.70	2.80	3.30	7.90	7.90
50th Percentile (ft)	2.30	2.95	2.95	3.55	8.30	8.25

	Top of Headrest Height		Bottom of Headrest Height		Top of Seat Height	
	Passenger Cars	Multipurpose Vehicles	Passenger Cars	Multipurpose Vehicles	Passenger Cars	Multipurpose Vehicles
Sample Size	368	328	368	328	368	328
Mean (ft)	4.14	4.92	3.54	4.31	3.45	4.20
Std. Deviation (ft)	0.20	0.29	0.20	0.33	0.19	0.32
5th Percentile (ft)	3.85	4.50	3.25	3.86	3.20	3.75
10th Percentile (ft)	3.90	4.60	3.30	3.95	3.25	3.85
15th Percentile (ft)	3.95	4.65	3.35	4.00	3.30	3.90
50th Percentile (ft)	4.15	4.85	3.55	4.30	3.45	4.15

Center of Headrest/Driver Eye Height (Average of top and bottom of headrest)		
	Passenger Cars	Multipurpose Vehicles
Sample Size	368	328
Mean (ft)	3.84	4.62
Std. Deviation (ft)	0.20	0.31
5th Percentile (ft)	3.55	4.18
10th Percentile (ft)	3.60	4.28
15th Percentile (ft)	3.65	4.33
50th Percentile (ft)	3.85	4.58

Driver Eye Height Comparison Between North Carolina and NCHRP 400 (Fambro et al., 1997)				
	Passenger Cars		Multipurpose Vehicles	
	North Carolina	NCHRP-400	North Carolina	NCHRP-400
Sample Size	368	875	328	629
Mean (ft)	3.84	3.77	4.62	4.86
Std. Deviation (ft)	0.20	0.18	0.31	0.43
5th Percentile (ft)	3.55	3.48	4.18	4.15
10th Percentile (ft)	3.60	3.55	4.28	4.28
15th Percentile (ft)	3.65	3.59	4.33	4.37
50th Percentile (ft)	3.85	–	4.58	–

Note: – parameters are not provided.

Table 4-3. Vehicle Fleet and Driver Eye Height Summary – Pennsylvania Data.

	Headlight Height		Taillight Height		Length from Bumper to Front of Driver Seat	
	Passenger Cars	Multipurpose Vehicles	Passenger Cars	Multipurpose Vehicles	Passenger Cars	Multipurpose Vehicles
Sample Size	309	295	309	295	309	295
Mean (ft)	2.29	2.86	2.98	3.54	8.41	8.50
Std. Deviation (ft)	0.26	0.29	0.29	0.39	0.43	0.47
5th Percentile (ft)	2.00	2.40	2.63	3.00	7.70	7.74
10th Percentile (ft)	2.05	2.55	2.70	3.11	7.88	8.00
15th Percentile (ft)	2.10	2.62	2.75	3.20	8.00	8.10
50th Percentile (ft)	2.25	2.82	2.91	3.50	8.40	8.44

	Top of Headrest Height		Bottom of Headrest Height		Top of Seat Height	
	Passenger Cars	Multipurpose Vehicles	Passenger Cars	Multipurpose Vehicles	Passenger Cars	Multipurpose Vehicles
Sample Size	309	295	309	295	309	295
Mean (ft)	4.19	4.84	3.60	4.19	3.51	4.07
Std. Deviation (ft)	0.26	0.33	0.26	0.32	0.27	0.32
5th Percentile (ft)	3.84	4.34	3.28	3.76	3.20	3.59
10th Percentile (ft)	3.90	4.50	3.32	3.86	3.26	3.79
15th Percentile (ft)	3.95	4.58	3.38	3.90	3.30	3.84
50th Percentile (ft)	4.15	4.80	3.54	4.15	3.45	4.00

Center of Headrest/Driver Eye Height (Average of top and bottom of headrest)		
	Passenger Cars	Multipurpose Vehicles
Sample Size	309	295
Mean (ft)	3.89	4.51
Std. Deviation (ft)	0.26	0.32
5th Percentile (ft)	3.56	4.05
10th Percentile (ft)	3.61	4.18
15th Percentile (ft)	3.67	4.24
50th Percentile (ft)	3.85	4.48

Driver Eye Height Comparison Between Michigan and NCHRP 400 (Fambro et al., 1997)				
	Passenger Cars		Multipurpose Vehicles	
	Michigan	NCHRP-400	Michigan	NCHRP-400
Sample Size	309	875	295	629
Mean (ft)	3.89	3.77	4.51	4.86
Std. Deviation (ft)	0.26	0.18	0.32	0.43
5th Percentile (ft)	3.56	3.48	4.05	4.15
10th Percentile (ft)	3.61	3.55	4.18	4.28
15th Percentile (ft)	3.67	3.59	4.24	4.37
50th Percentile (ft)	3.85	–	4.48	–

Note: – parameters are not provided.

A.1.1 Field Studies of Vehicle Acceleration and Deceleration Performance

A.1.1.1 Site Summary

This section presents the general details about location and design features of each of the entrance and exit ramps where field data collection was carried out. Table 4-4 provides the location of each of the entrance ramps included in the study. At site CA-5, the ramp meter was active during the entire period of data collection, while at sites CA-8 and CA-10, data were collected during both periods − when the ramp meter was active and when it was inactive. Table 4-5 presents details of the design features of each of the entrance ramps while Table 4-6 details speed limit and design speed information for each entrance ramp and associated mainline segment. Similarly, Table 4-7 presents the location of each of the exit ramps included in the study. Table 4-8 present the details of the design features of each of the exit ramps and Table 4-9 presents pertinent speed-related information for each exit ramp and associated mainline segment.

Table 4-4. Entrance Ramp Sites.

Ramp ID	State	Freeway	Crossroad	Direction
CA-1	CA	US-101	S Rancho Rd	SB Entrance
CA-2	CA	US-101	Camarillo Springs Rd	NB Entrance
CA-3	CA	US-101	Santa Rosa Rd	NB Entrance
CA-4	CA	I-210	Sunland Blvd	SB Entrance
CA-5	CA	I-10	Peck Rd	WB Entrance
CA-6	CA	I-10	Valley Blvd	WB Entrance
CA-7	CA	I-710	Long Beach Blvd	SB Entrance
CA-8	CA	I-710	E Washington Blvd	NB Entrance
CA-9	CA	I-10	Valley Blvd	EB Entrance
CA-10	CA	SR-60	Paramount Blvd	EB Entrance
MI-1	MI	I-69	S Irish Rd	NB Entrance
MI-2	MI	I-69	Webster Rd	NB Entrance
MI-3	MI	I-69	SR-52	SB Entrance
MI-4	MI	I-94	SR-140	WB Entrance
MI-5	MI	I-96	Plainfield Ave NE	WB Entrance
MI-6	MI	I-96	E Grand River Ave	EB Entrance
MI-7	MI	I-96	E Beltline Ave NE	WB Entrance
MI-8	MI	I-94	W Columbia Ave	EB Entrance
NC-1	NC	I-277	3rd St	SB Entrance
NC-2	NC	I-277	N Brevard St	SB Entrance
NC-3	NC	I-485	S Tyron St	SB Entrance
NC-4	NC	I-85	York Rd	NB Entrance
NC-5	NC	I-77	SR-150	NB Entrance
NC-6	NC	I-77	Amity Hill Rd	NB Entrance
NC-7	NC	US-74	US-601	WB Entrance
NC-8	NC	I-85	Bessemer City Rd	NB Entrance
PA-1	PA	US-220	Plank Rd	EB Entrance
PA-2	PA	I-80	Appalachain Trwy	EB Entrance
PA-3	PA	US-220	SR-4007	SB Entrance
PA-4	PA	I-99	Benner Pike	SB Entrance
PA-5	PA	I-99	Waddle Rd	SB Entrance
PA-6	PA	I-99	Waddle Rd	NB Entrance
PA-7	PA	US-322	Boalsburg Rd	WB Entrance
PA-8	PA	US-322	SR-655	SB Entrance
PA-9	PA	US-322	Old US Hwy 322	SB Entrance

Table 4-5. Entrance Ramp Design Features.

Ramp ID	Ramp Type	Merge Type	Controlling Feature	Radius of Controlling Curve (ft)	Grade	Acceleration Lane Length (ft)	SCL Length (ft)
CA-1	Loop	Parallel	Curve	144	2%	619	296
CA-2	Diamond	Parallel	Curve	151	3%	480	480
CA-3	Loop	Tapered	Curve	143	-4%	505	343
CA-4	Loop	Parallel	Curve	189	5%	692	251
CA-5	Diamond	Parallel	Curve	208	2%	550	384
CA-6	Loop	Tapered	Curve	123	3%	866	814
CA-7	Diamond	Parallel	Curve	487	-1%	540	303
CA-8	Diamond	Parallel	Curve	202	4%	138	65
CA-9	Loop	Tapered	Curve	126	2%	479	200
CA-10	Loop	Tapered	Curve	183	-4%	321	216
MI-1	Loop	Parallel	Curve	245	2%	1,233	1190
MI-2	Diamond	Tapered	Crossroad	–	-2%	1,951	235
MI-3	Loop	Tapered	Curve	278	-2%	542	276
MI-4	Diamond	Parallel	Crossroad	–	4%	2,159	424
MI-5	Diamond	Parallel	Curve	637	-2%	1,196	875
MI-6	Loop	Parallel	Curve	235	2%	977	951
MI-7	Loop	Parallel	Curve	368	-2%	475	475
MI-8	Diamond	Parallel	Curve	330	-1%	1,607	288
NC-1	Diamond	Tapered	Crossroad	–	2%	815	79
NC-2	Diamond	Tapered	Curve	354	-5%	585	527
NC-3	Loop	Parallel	Curve	208	-4%	880	756
NC-4	Loop	Parallel	Curve	275	-4%	835	695
NC-5	Diamond	Tapered	Crossroad	–	-2%	1,341	203
NC-6	Diamond	Parallel	Crossroad	–	-2%	1,403	142
NC-7	Loop	Parallel	Curve	260	4%	1,420	1420
NC-8	Loop	Parallel	Curve	330	-2%	1,059	953
PA-1	Loop	Tapered	Curve	294	4%	920	114
PA-2	Loop	Parallel	Curve	443	4%	1,269	674
PA-3	Loop	Tapered	Curve	745	0%	462	53
PA-4	Loop	Parallel	Curve	477	2%	1,283	307
PA-5	Diamond	Parallel	Crossroad	–	-2%	2,948	1076
PA-6	Diamond	Tapered	Crossroad	–	-4%	2,132	42
PA-7	Diamond	Tapered	Crossroad	–	5%	2,289	109
PA-8	Diamond	Parallel	Crossroad	–	2%	1,724	495
PA-9	Loop	Parallel	Curve	282	-2%	1,311	386

Note: – parameter is not pertinent for crossroad terminal.

Table 4-6. Freeway Design, Posted Speeds, and Ramp Design Speeds for Entrance Ramps.

Ramp ID	Posted Speed Limit on Freeway (mph)	Freeway Design Speed (mph)	Ramp Design Speed (mph)
CA-1	65	65	25
CA-2	65	65	25
CA-3	65	65	25
CA-4	65	65	25
CA-5	65	65	25
CA-6	65	65	20
CA-7	55	55	40
CA-8	65	65	25
CA-9	65	65	20
CA-10	65	65	25
MI-1	70	70	30
MI-2	70	70	Stop condition
MI-3	75	75	30
MI-4	70	70	Stop condition
MI-5	70	70	45
MI-6	70	70	30
MI-7	70	70	35
MI-8	70	70	35
NC-1	50	50	Stop condition
NC-2	50	50	35
NC-3	70	70	25
NC-4	65	65	30
NC-5	65	65	Stop condition
NC-6	65	65	Stop condition
NC-7	65	65	30
NC-8	65	65	35
PA-1	65	65	30
PA-2	70	70	40
PA-3	55	55	45
PA-4	70	70	40
PA-5	65	65	Stop condition
PA-6	65	65	Stop condition
PA-7	55	55	Stop condition
PA-8	65	65	Stop condition
PA-9	55	55	30

Table 4-7. Exit Ramp Sites.

Ramp ID	State	Freeway	Crossroad	Direction
CA-11	CA	US-101	S Westlake Blvd	WB Exit
CA-12	CA	US-101	Santa Rosa Rd	EB Exit
CA-13	CA	I-5	The Old Rd	SB Exit
CA-14	CA	I-5	Lake Hughes Rd	NB Exit
CA-15	CA	I-10	Peck Rd	WB Exit
CA-16	CA	SR-2	Camino San Rafael	SB Exit
MI-9	MI	I-69	S Irish Rd	NB Exit
MI-10	MI	I-69	Webster Rd	NB Exit
MI-11	MI	I-69	SR-52	SB Exit
MI-12	MI	I-94	SR-140	EB Exit
MI-13	MI	I-96	E Grand River Ave	EB Exit
MI-14	MI	I-94	Partello Rd	WB Exit
MI-15	MI	I-94	SR-99	WB Exit
MI-16	MI	I-94	N Parma Rd	WB Exit
NC-9	NC	I-77	W Woodlawn Rd	NB Exit
NC-10	NC	I-40	1st St W	EB Exit
NC-11	NC	I-40	Rock Barn Rd NE	WB Exit
NC-12	NC	I-85	N Chester St	NB Exit
NC-13	NC	I-85	S Main St	NB Exit
NC-14	NC	I-85	Bessemer City Rd	NB Exit
NC-15	NC	I-77	SR-150	SB Exit
NC-16	NC	I-85	Bessemer City Rd	SB Exit
PA-10	PA	I-99	SR-220	NB Exit
PA-11	PA	US-220	Plank Rd	EB Exit
PA-12	PA	I-99	Frankstown Rd	SB Exit
PA-13	PA	I-80	Beech Creek Rd	WB Exit
PA-14	PA	US-220	SR-144	NB Exit
PA-15	PA	I99	Shiloh Rd	SB Exit
PA-16	PA	US-322	Boalsburg Rd	EB Exit
PA-17	PA	US-322	Old US Hwy 322	SB Exit

Table 4-8. Exit Ramp Design Features.

Ramp ID	Ramp Type	Diverge Type	Controlling Feature	Radius of Controlling Curve (ft)	Grade	Deceleration Lane Length (ft)	SCL Length (ft)
CA-11	Diamond	Tapered	Curve	440	1%	791	0
CA-12	Loop	Tapered	Curve	162	0%	335	0
CA-13	Diamond	Tapered	Curve	109	-1%	599	0
CA-14	Diamond	Tapered	Crossroad	–	-2%	1,387	0
CA-15	Loop	Parallel	Curve	128	-2%	889	812
CA-16	Diamond	Tapered	Crossroad	–	2%	915	0
MI-9	Diamond	Parallel	Curve	334	-1%	1,958	524
MI-10	Loop	Tapered	Curve	284	2%	733	101
MI-11	Diamond	Tapered	Curve	685	2%	1,812	122
MI-12	Loop	Parallel	Curve	246	-4%	1,000	834
MI-13	Diamond	Tapered	Curve	561	-2%	302	71
MI-14	Loop	Tapered	Curve	241	2%	200	0
MI-15	Loop	Tapered	Curve	243	1%	300	36
MI-16	Diamond	Tapered	Crossroad	–	-2%	910	53
NC-9	Loop	Parallel	Curve	234	4%	404	313
NC-10	Loop	Parallel	Curve	231	4%	410	406
NC-11	Diamond	Tapered	Crossroad	–	2%	750	24
NC-12	Diamond	Parallel	Curve	561	-2%	431	334
NC-13	Loop	Parallel	Curve	231	3%	730	671
NC-14	Diamond	Parallel	Curve	587	2%	567	515
NC-15	Diamond	Tapered	Crossroad	–	2%	980	88
NC-16	Loop	Parallel	Curve	213	4%	741	635
PA-10	Loop	Tapered	Curve	254	-1%	335	15
PA-11	Diamond	Parallel	Curve	673	-5%	740	145
PA-12	Loop	Parallel	Curve	322	4%	855	135
PA-13	Diamond	Parallel	Crossroad	–	2%	1,826	462
PA-14	Diamond	Tapered	Crossroad	–	2%	1,385	21
PA-15	Loop	Parallel	Curve	370	4%	1,220	549
PA-16	Diamond	Tapered	Crossroad	–	-4%	1,870	37
PA-17	Loop	Parallel	Curve	284	2%	964	455

Note: – parameter is not pertinent for crossroad terminal.

Table 4-9. Freeway Design, Posted Speeds, and Ramp Design Speeds for Exit Ramps.

Ramp ID	Posted Speed Limit on Freeway (mph)	Freeway Design Speed (mph)	Ramp Design Speed (mph)
CA-11	65	65	35
CA-12	65	65	25
CA-13	65	65	20
CA-14	65	65	Stop condition
CA-15	65	65	20
CA-16	65	65	Stop condition
MI-9	70	70	35
MI-10	70	70	30
MI-11	75	75	45
MI-12	70	70	30
MI-13	70	70	40
MI-14	70	70	30
MI-15	70	70	30
MI-16	70	70	Stop condition
NC-9	55	55	30
NC-10	65	65	30
NC-11	65	65	Stop condition
NC-12	65	65	40
NC-13	60	60	30
NC-14	65	65	40
NC-15	65	65	Stop condition
NC-16	65	65	25
PA-10	65	65	30
PA-11	65	65	45
PA-12	65	65	35
PA-13	70	70	Stop condition
PA-14	55	55	Stop condition
PA-15	65	65	35
PA-16	55	55	Stop condition
PA-17	55	55	30

A.1.1.2 Entrance Ramps

This section provides the State-by-State plots of various design features for entrance ramps based on the results from the field data collection.

A.1.1.2.1 Merge Location

This section provides the State-by-State plots of various design features for entrance ramps based on the results from the field data collection. Figure 4-1 through Figure 4-4 provide a State-by-State comparison of summary information as to the merge locations that were observed, expressed as a percentage of the SCL that was utilized. Table 4-10 shows the percentage of late merge maneuvers that were observed at each study location along with the summary characteristics for each ramp, including the difference between the acceleration lane length and the recommended values from the 2018 Green Book. Negative values indicate that the actual length is less than the recommendation value.

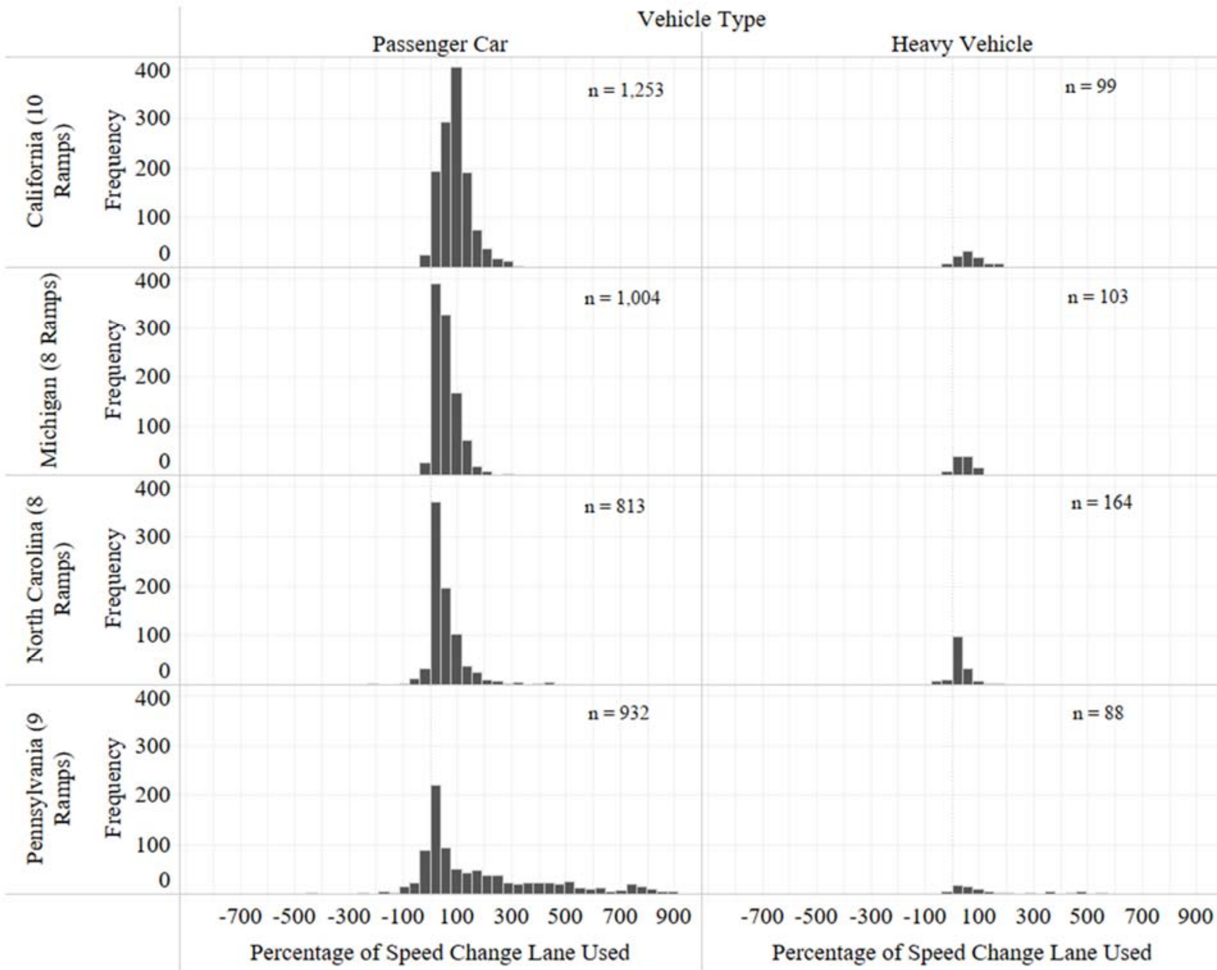


Figure 4-1. Percentage of SCL Used by Vehicle Type (Passenger Car versus Heavy Vehicle) for Entrance Ramps.

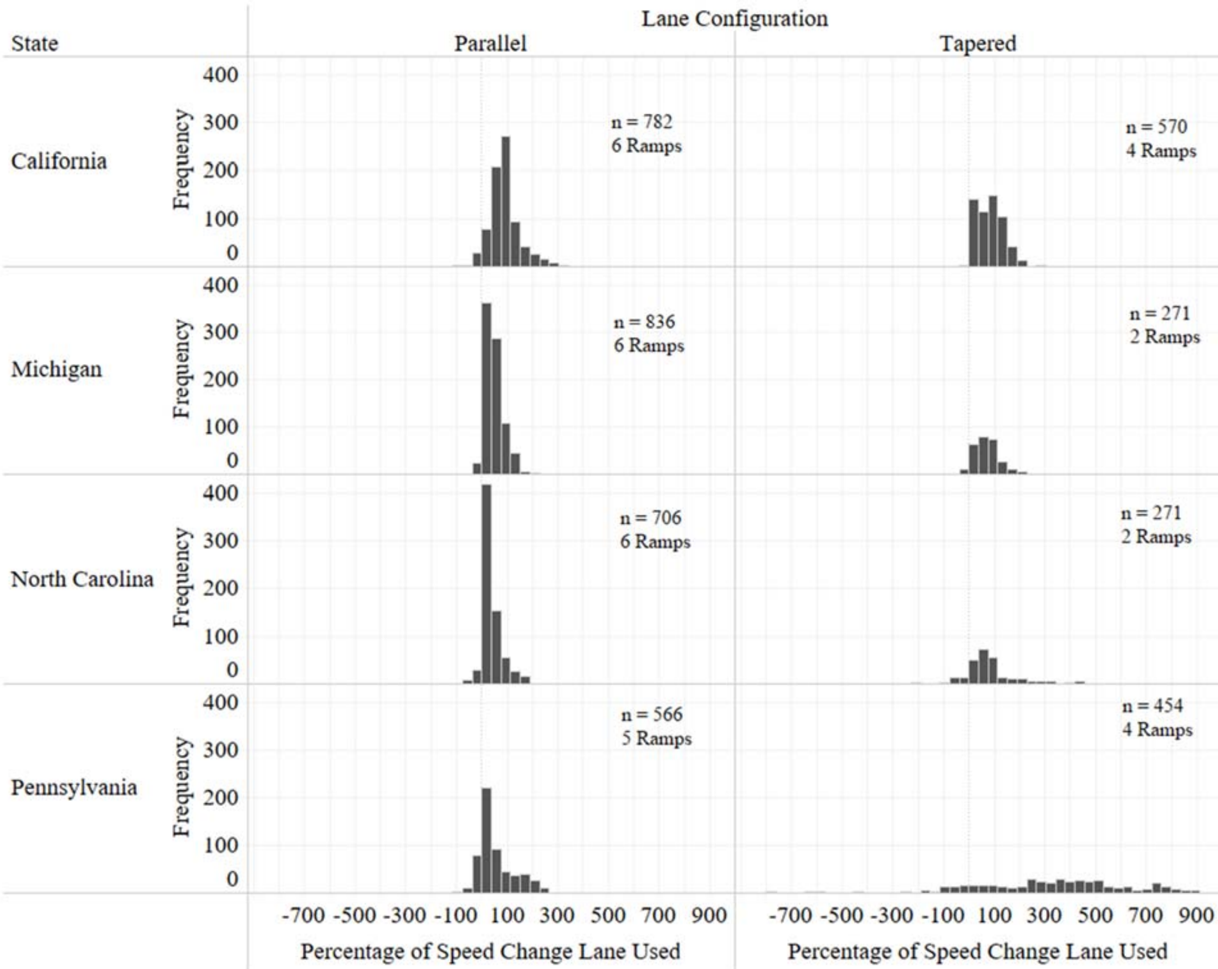


Figure 4-2. Percentage of SCL Used by Lane Configuration (Parallel versus Tapered) for Entrance Ramps.

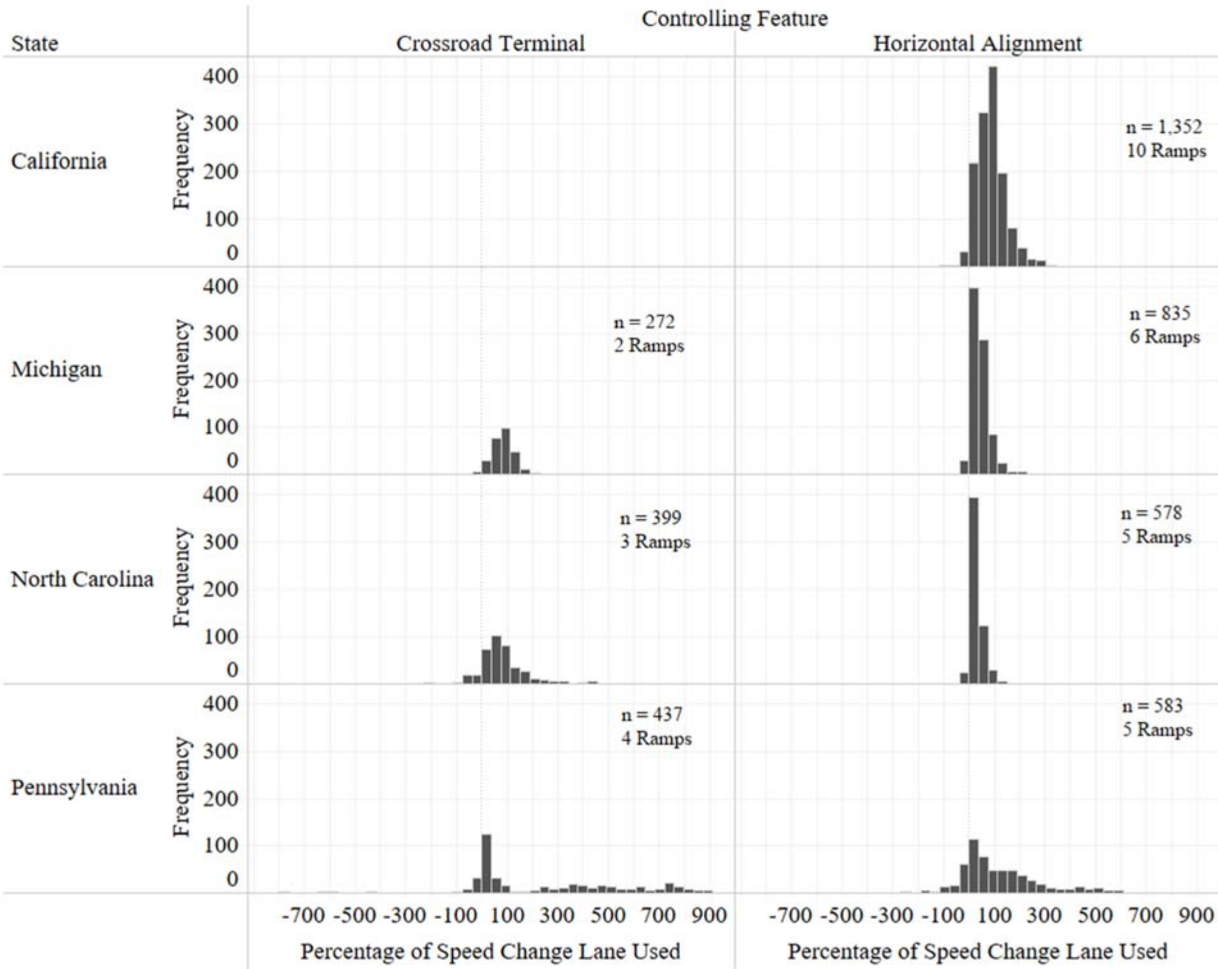


Figure 4-3. Percentage of SCL Used by Controlling Feature (Crossroad Terminal versus Curve) for Entrance Ramps.

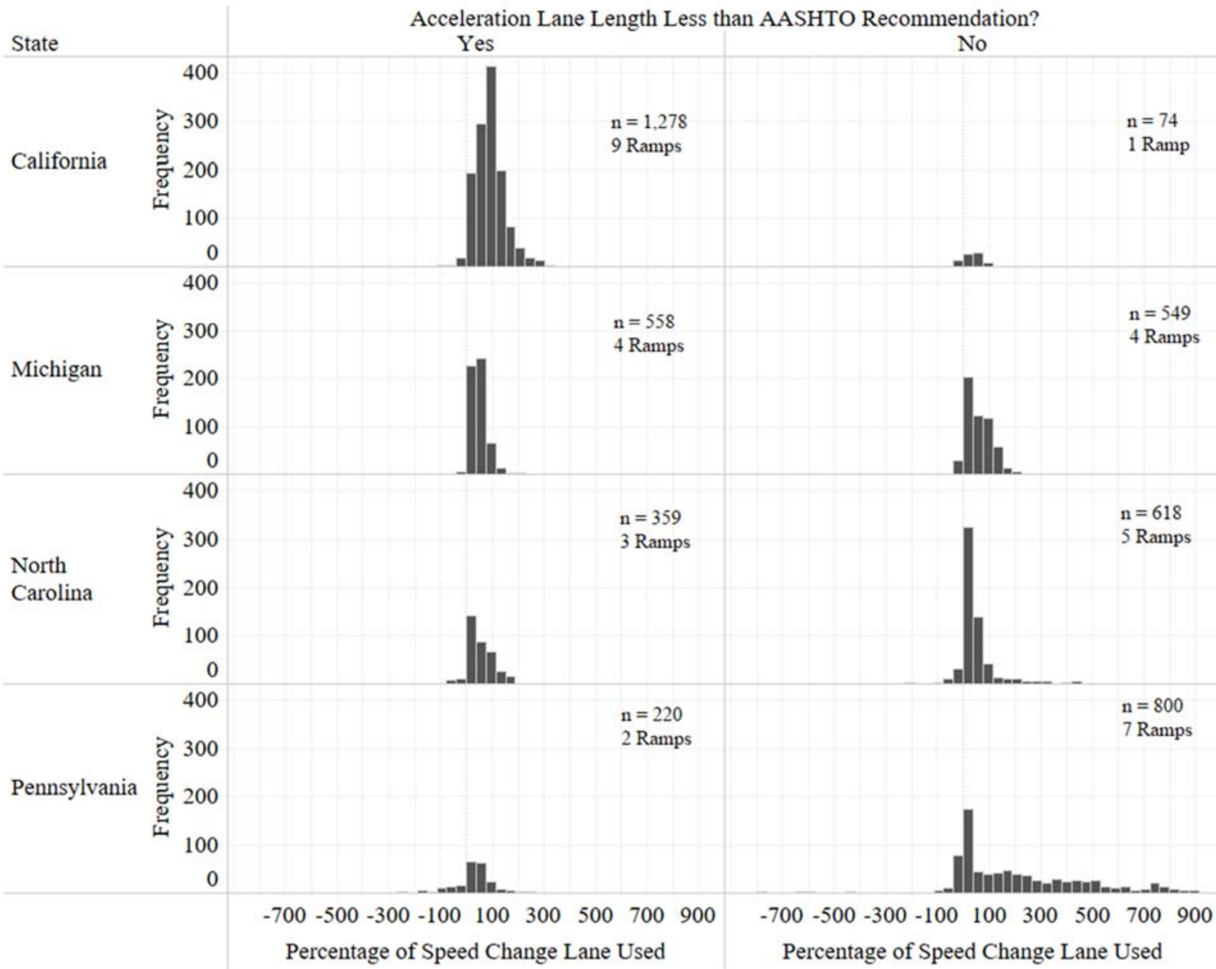


Figure 4-4. Percentage of SCL Used based on Minimum Criteria of Acceleration Lane Length from AASHTO Green Book (2018).

Table 4-10. Percentage of Late Merges Observed by Site.

Ramp ID	Merge Type	SCL Length (ft)	Acceleration Lane Length (ft)	Green Book Minimum Acceleration Lane Length (ft)	Difference in Acceleration Lane Length Compared to Minimum (percent)	Number of Observations	Percentage of Late Merges Observed
CA-1	Parallel	296	619	1,220	-49	157	54
CA-2	Parallel	480	480	2,013	-76	127	9
CA-3	Tapered	343	505	610	-17	118	31
CA-4	Parallel	251	692	2,745	-75	151	50
CA-5	Parallel	384	550	1,220	-55	153	14
CA-6	Tapered	814	866	1,310	-34	134	0
CA-7	Parallel	303	540	320	69	94	18
CA-8	Parallel	65	138	2,074	-93	120	57
CA-9	Tapered	200	479	1,310	-63	160	50
CA-10	Tapered	216	321	732	-56	158	63
MI-1	Parallel	1190	1,233	1,350	-9	121	0
MI-2	Tapered	235	1,951	1,620	20	125	38
MI-3	Tapered	276	542	1,510	-64	146	13
MI-4	Parallel	424	2,159	1,620	33	147	37
MI-5	Parallel	875	1,196	820	46	153	0
MI-6	Parallel	951	977	1,350	-28	145	1
MI-7	Parallel	475	475	1,230	-61	146	10
MI-8	Parallel	288	1,607	1,230	31	124	19
NC-1	Tapered	79	815	720	13	142	42
NC-2	Tapered	527	585	175	234	120	1
NC-3	Parallel	756	880	852	3	119	4
NC-4	Parallel	695	835	672	24	115	4
NC-5	Tapered	203	1,341	1,410	-5	129	10
NC-6	Parallel	142	1,403	1,410	0	128	35
NC-7	Parallel	1420	1,420	1,904	-25	102	0
NC-8	Parallel	953	1,059	1,000	6	122	3
PA-1	Tapered	114	920	1,904	-52	104	20
PA-2	Parallel	674	1,269	2,000	-37	116	3
PA-3	Tapered	53	462	150	208	124	94
PA-4	Parallel	307	1,283	1,000	28	130	94
PA-5	Parallel	1076	2,948	1,410	109	106	0
PA-6	Tapered	42	2,132	846	152	107	85
PA-7	Tapered	109	2,289	960	138	120	98
PA-8	Parallel	495	1,724	1,410	22	104	2
PA-9	Parallel	386	1,311	670	96	110	0

Note: SCL length is measured from painted nose to start of taper. Acceleration lane length is measured from controlling feature to start of taper.

A.1.1.2.2 Merge Speed

Figure 4-5 through Figure 4-10 show box plots of merge speed differentials (i.e., the difference between merge speeds and mainline speeds on the rightmost lane) by State. A negative speed differential indicates drivers merged at a lower speed than the mainline operating speed. Figure 4-11 shows the comparison of merge speeds between field observations and Green Book (AASHTO, 2018) values by State.

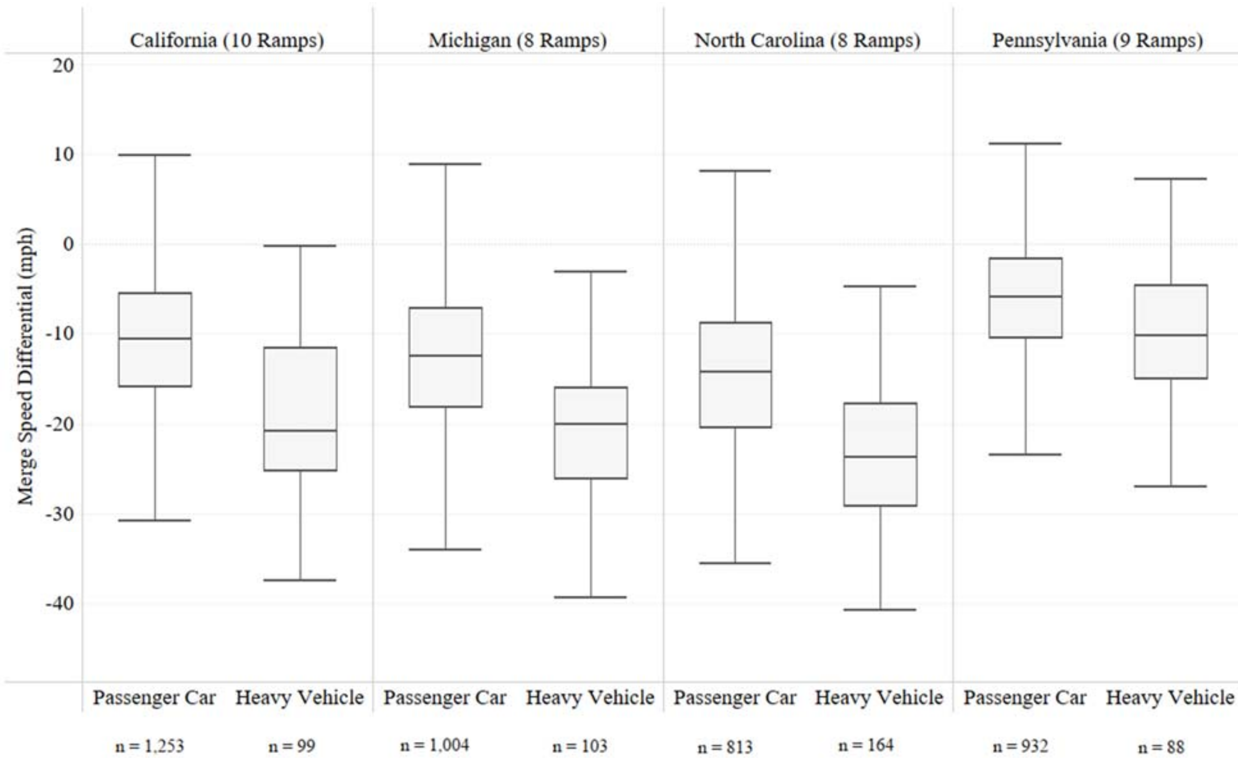


Figure 4-5. Merge Speed Differential by Vehicle Type (Passenger Car versus Heavy Vehicle).

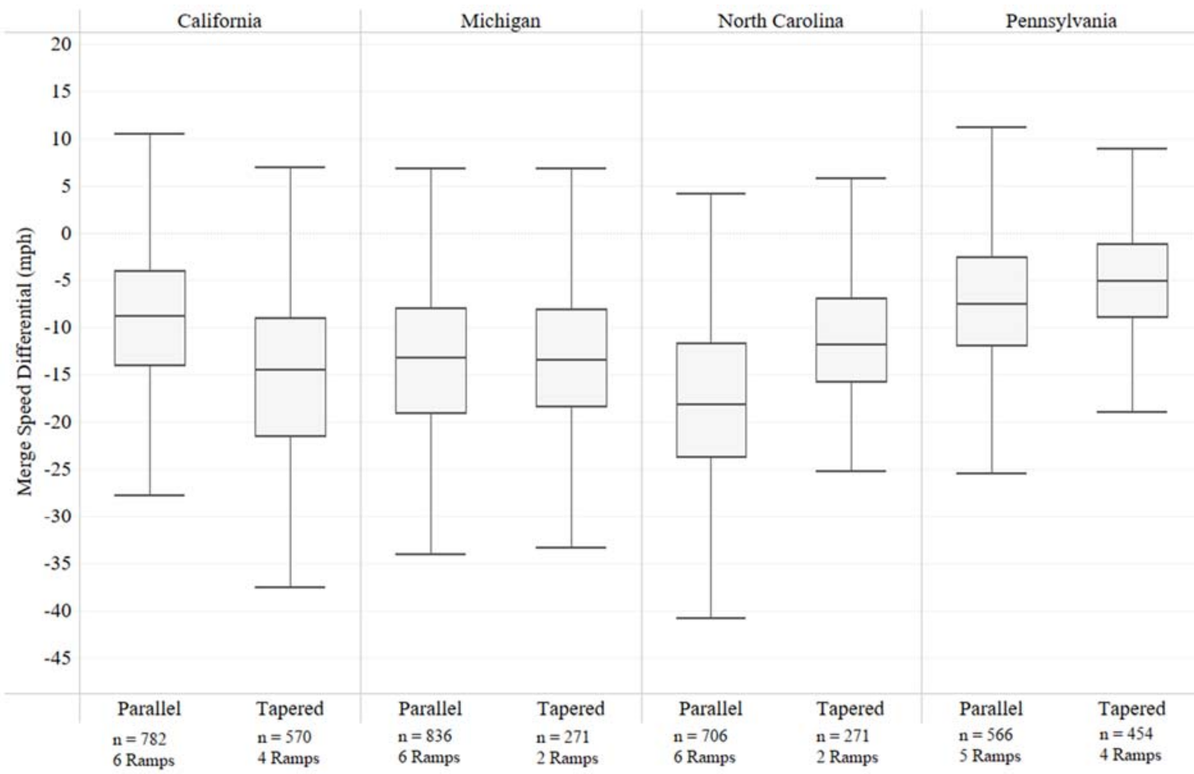


Figure 4-6. Merge Speed Differential by Lane Configuration (Parallel versus Tapered).

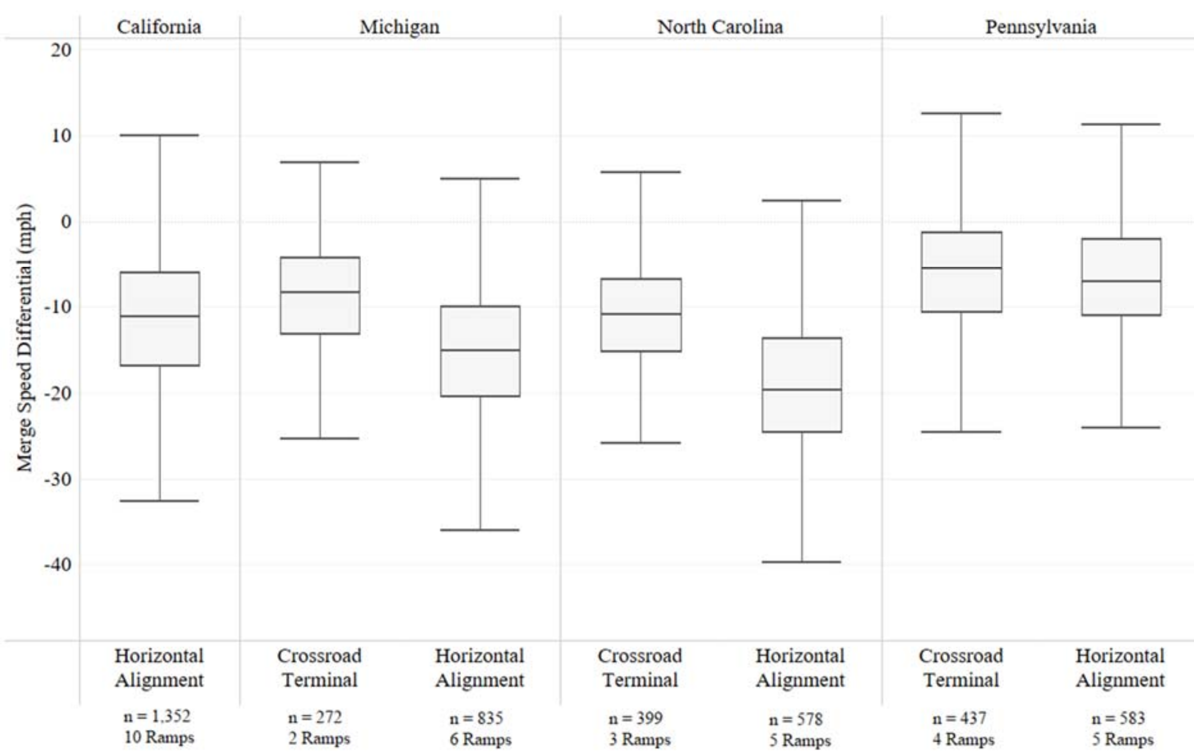


Figure 4-7. Merge Speed Differential by Controlling Feature (Crossroad Terminal versus Curve).

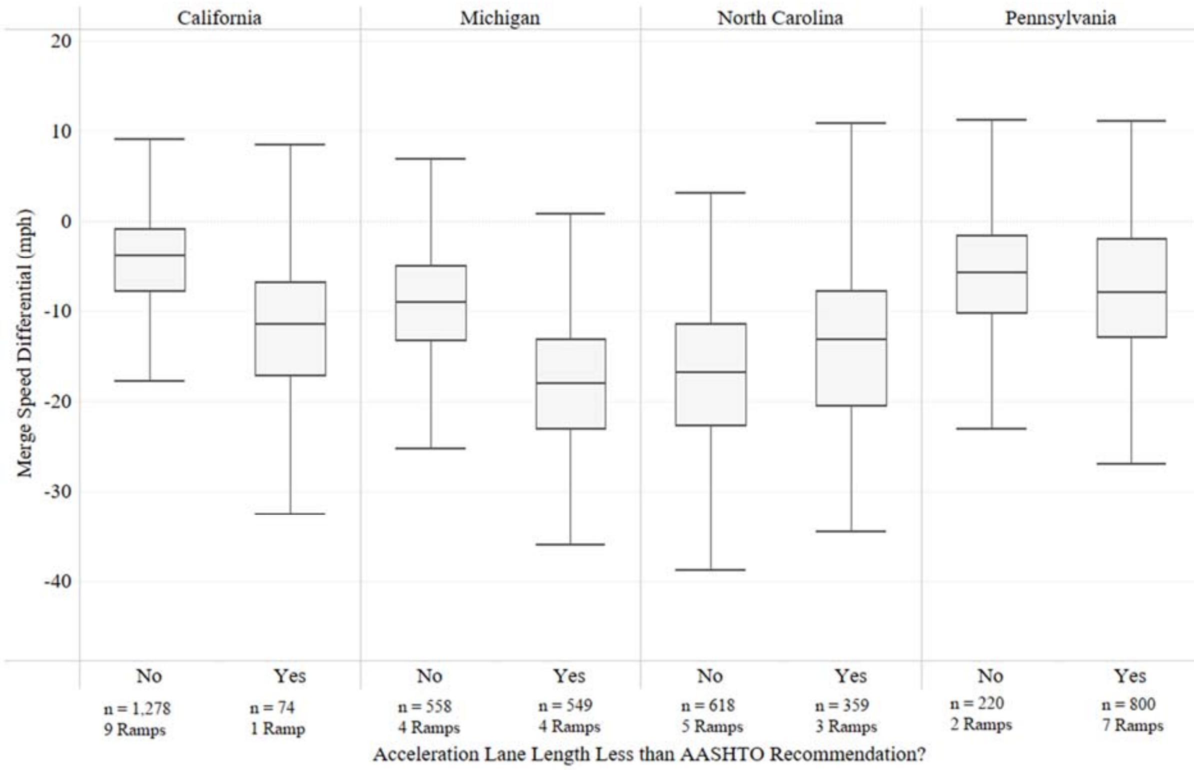


Figure 4-8. Merge Speed Differential based on Minimum Criteria of Acceleration Lane Length from AASHTO Green Book (2018).

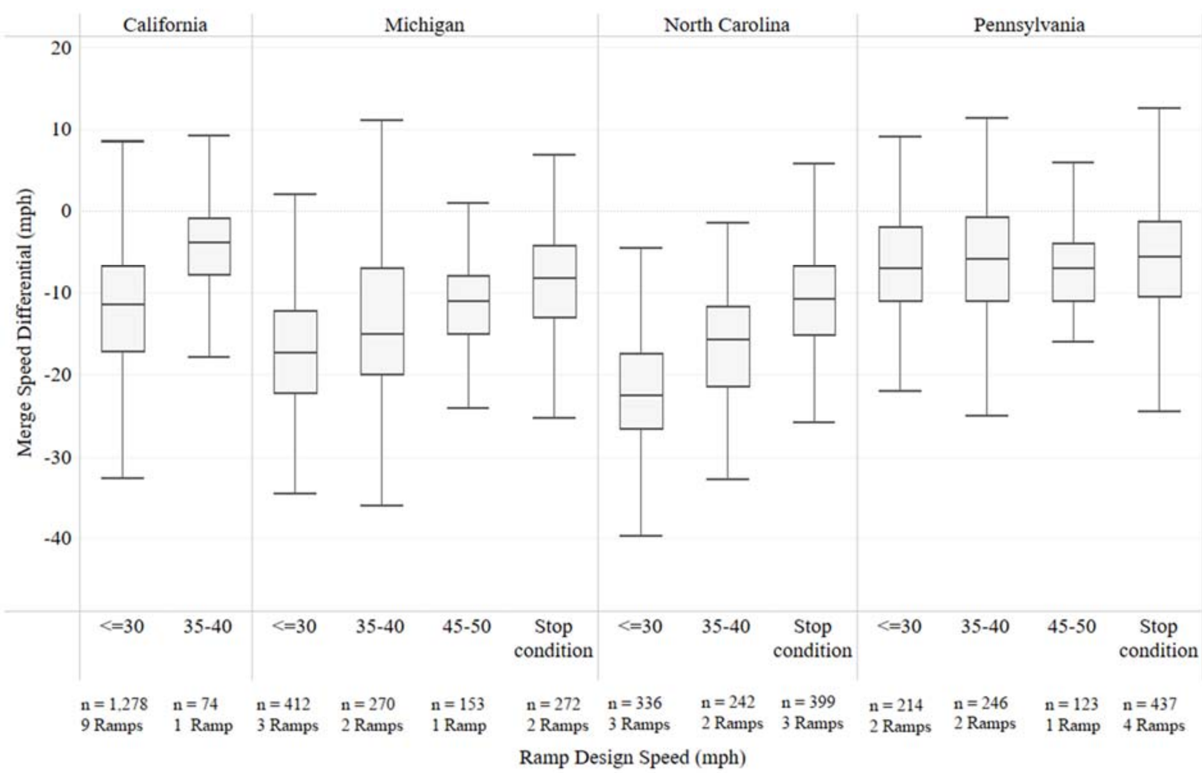


Figure 4-9. Merge Speed Differential based on Ramp Design Speed.

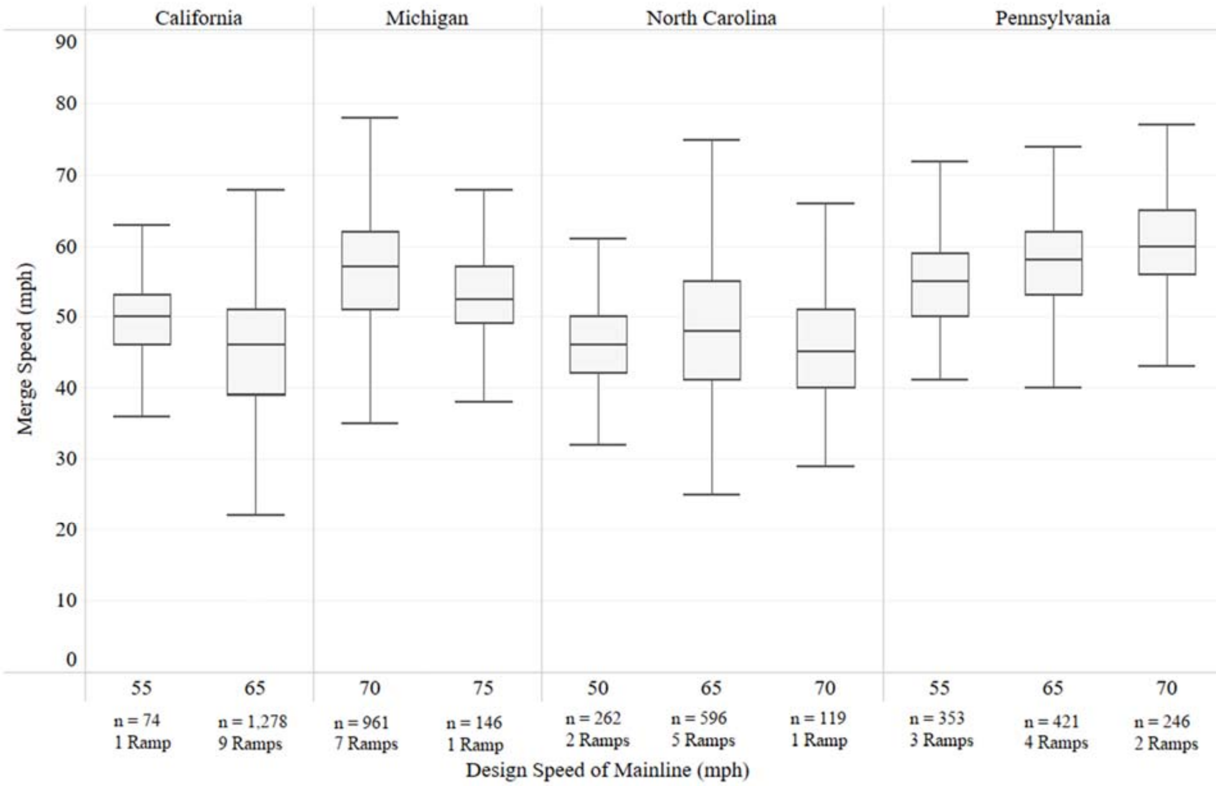


Figure 4-10. Merge Speed with Respect to Mainline Design Speed.

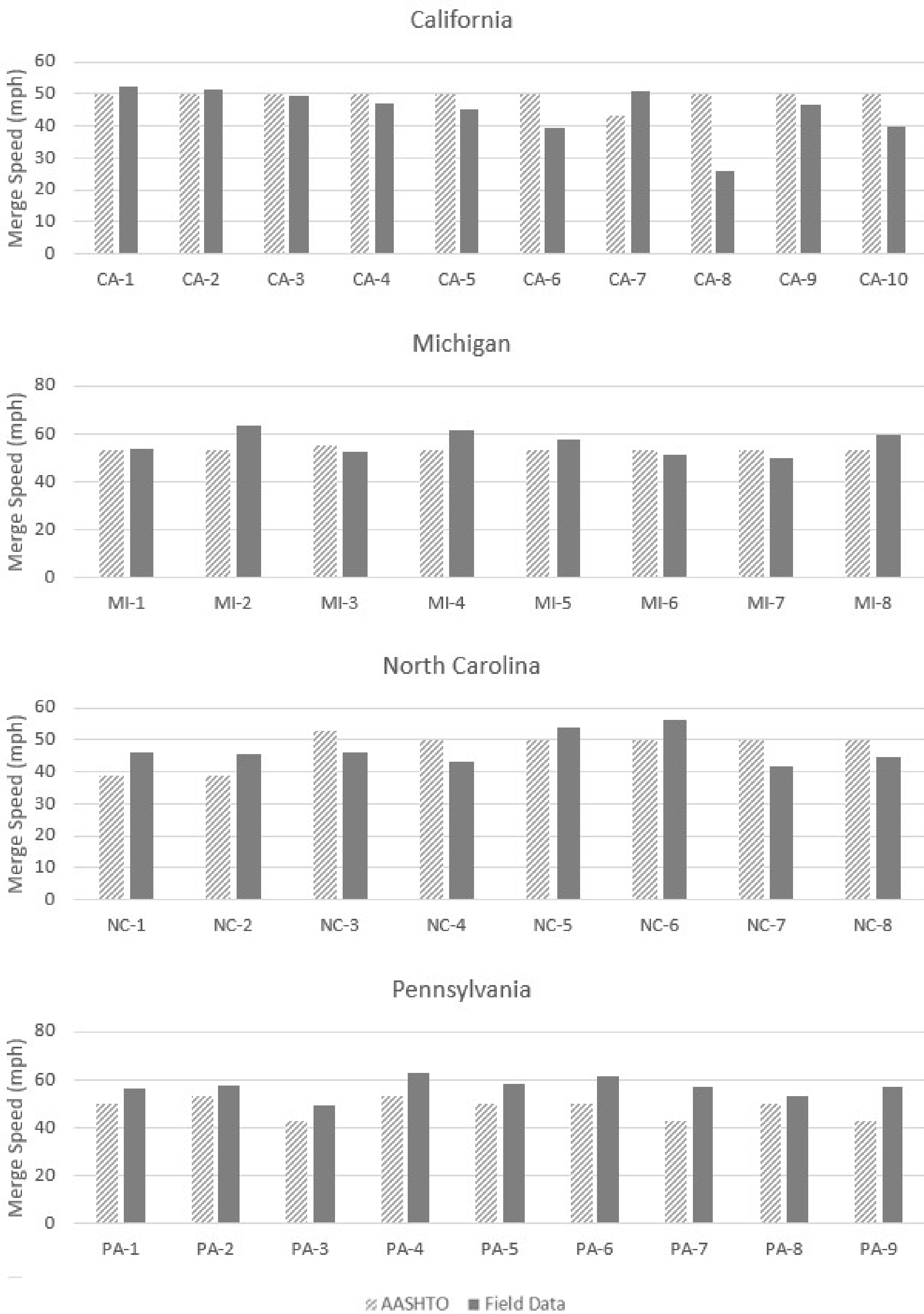


Figure* 4-11. *Merge Speed – Comparison between AASHTO assumed Values and Field Observations.

A.1.1.2.3 Speed at Controlling Feature

Figure 4-12 and Figure 4-13 show box plots of speed differentials at the point of curvature (PC) by vehicle type and based on minimum values of acceleration lane length from the 2018 Green Book, respectively.

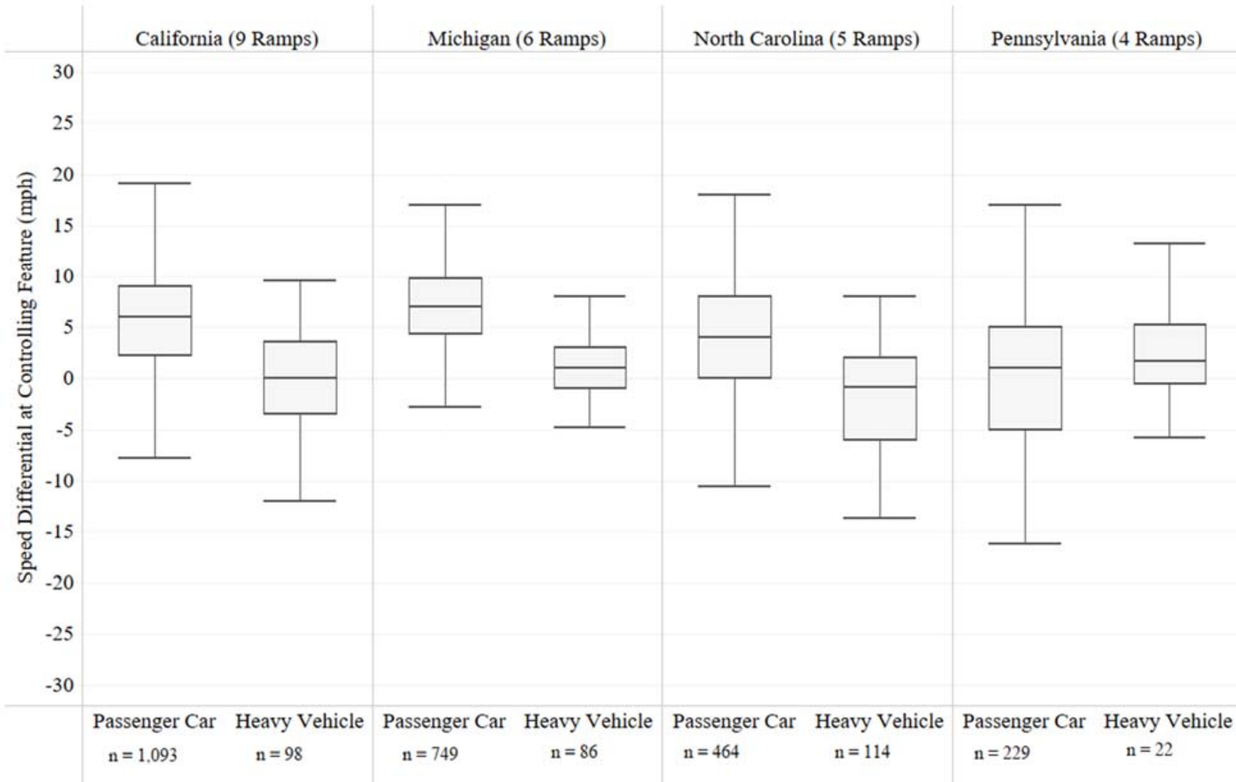


Figure 4-12. Speed Differential at Controlling Feature by Vehicle Type (Passenger Car versus Heavy Vehicle) for Entrance Ramps.

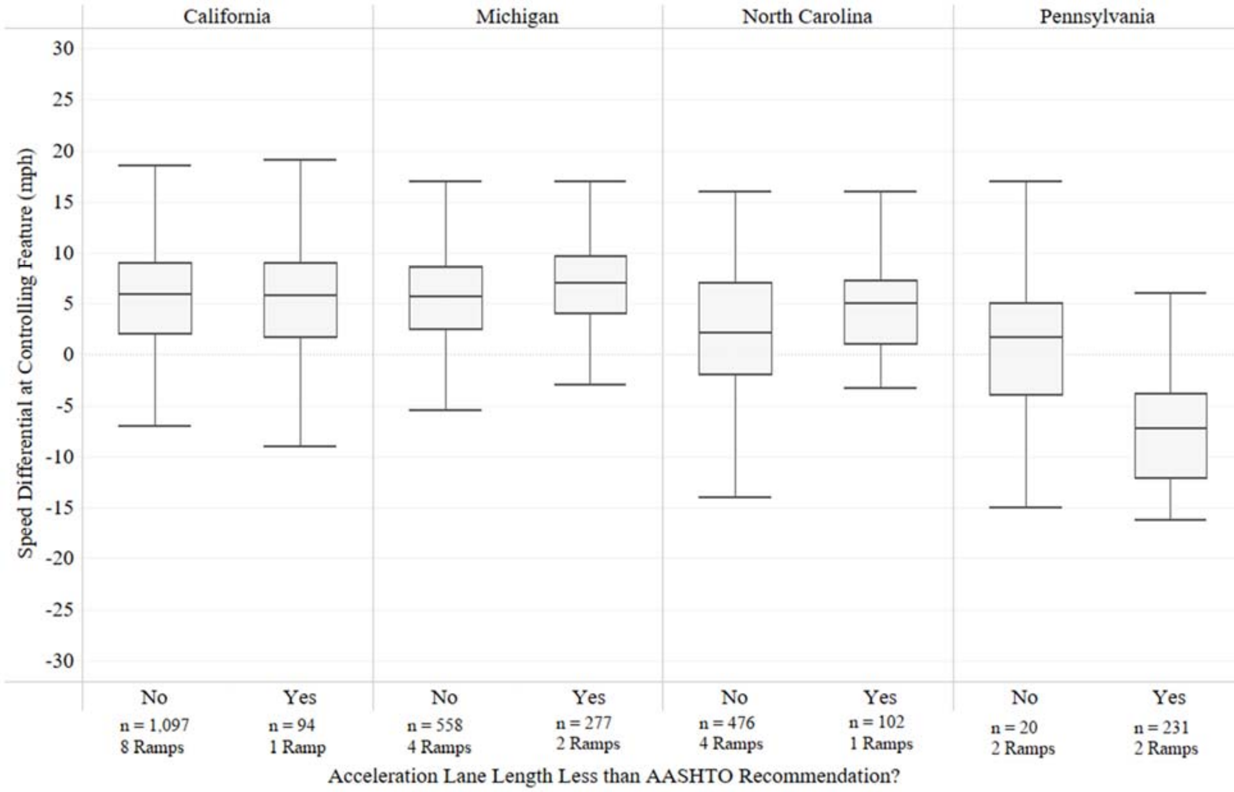


Figure 4-13. Speed Differential at Controlling Feature based on Minimum Criteria of Acceleration Lane Length from AASHTO Green Book (2018).

A.1.1.2.4 Acceleration Rate

Figure 4-14 through Figure 4-17 show acceleration rates based on different site characteristics and vehicle types. There are two types of acceleration rates presented in each figure—average and maximum acceleration rates. The average acceleration rate was calculated based on the initial speed (i.e., speeds at controlling feature if available, otherwise the first speed recorded from LiDAR), the final speed (i.e., speeds when left wheels touch the line separating the mainline and ramp traffic), and the distance between initial and final speeds. The maximum acceleration rates represent the highest average value across all 50-ft intervals at a given site. Figure 4-18 shows comparison of acceleration rates between Green Book values (AASHTO, 2018) and field observation with respect to each site.

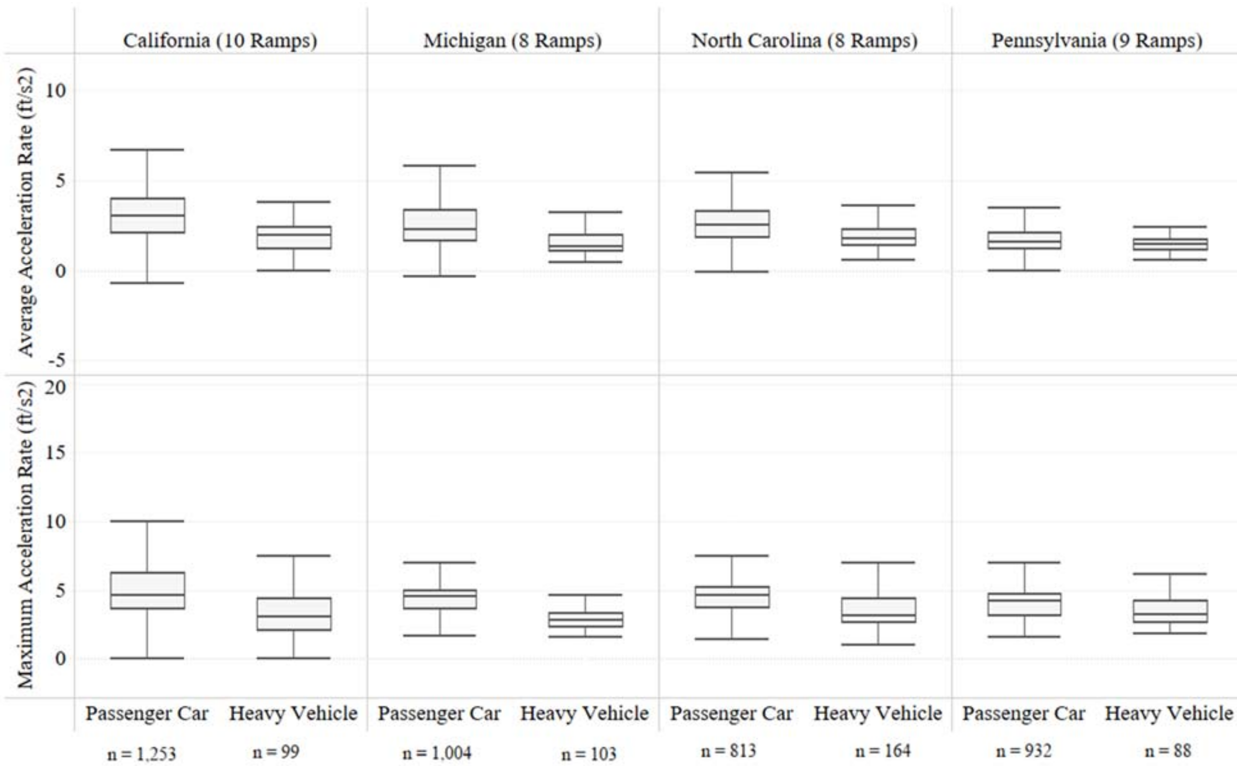


Figure 4-14. Acceleration Rate by Vehicle Type (Passenger Car versus Heavy Vehicle).

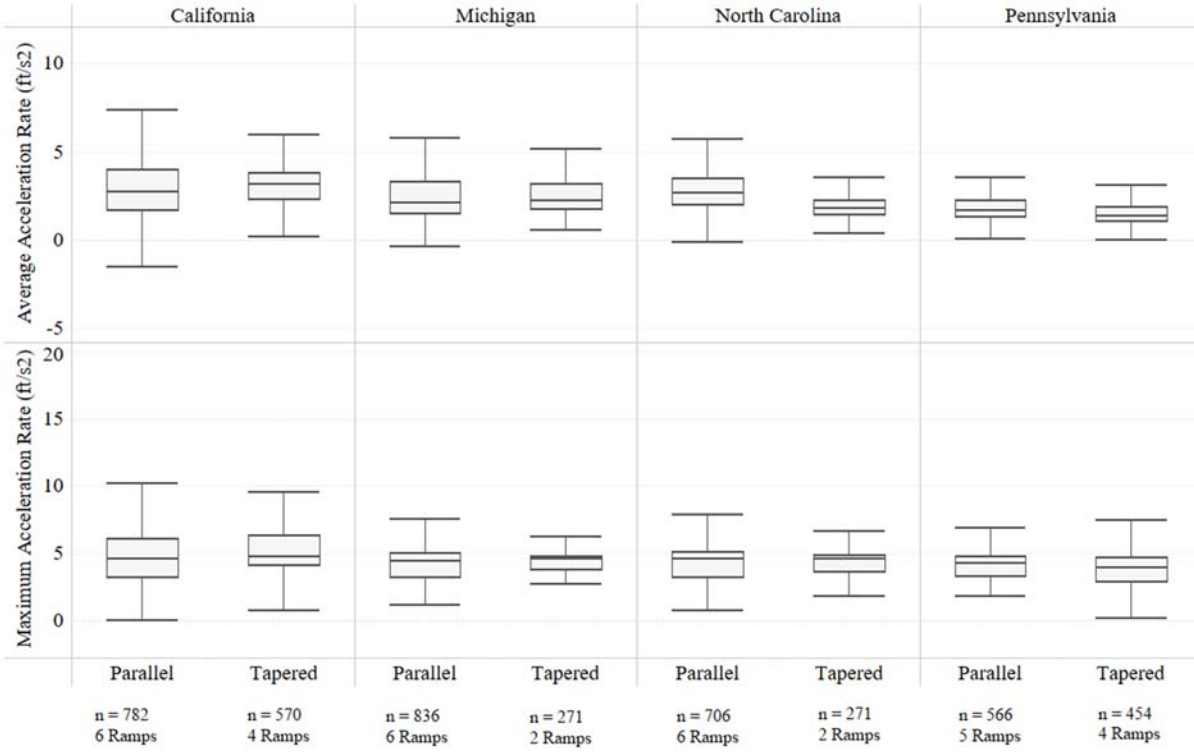


Figure 4-15. Acceleration Rate by Lane Configuration (Parallel versus Tapered).

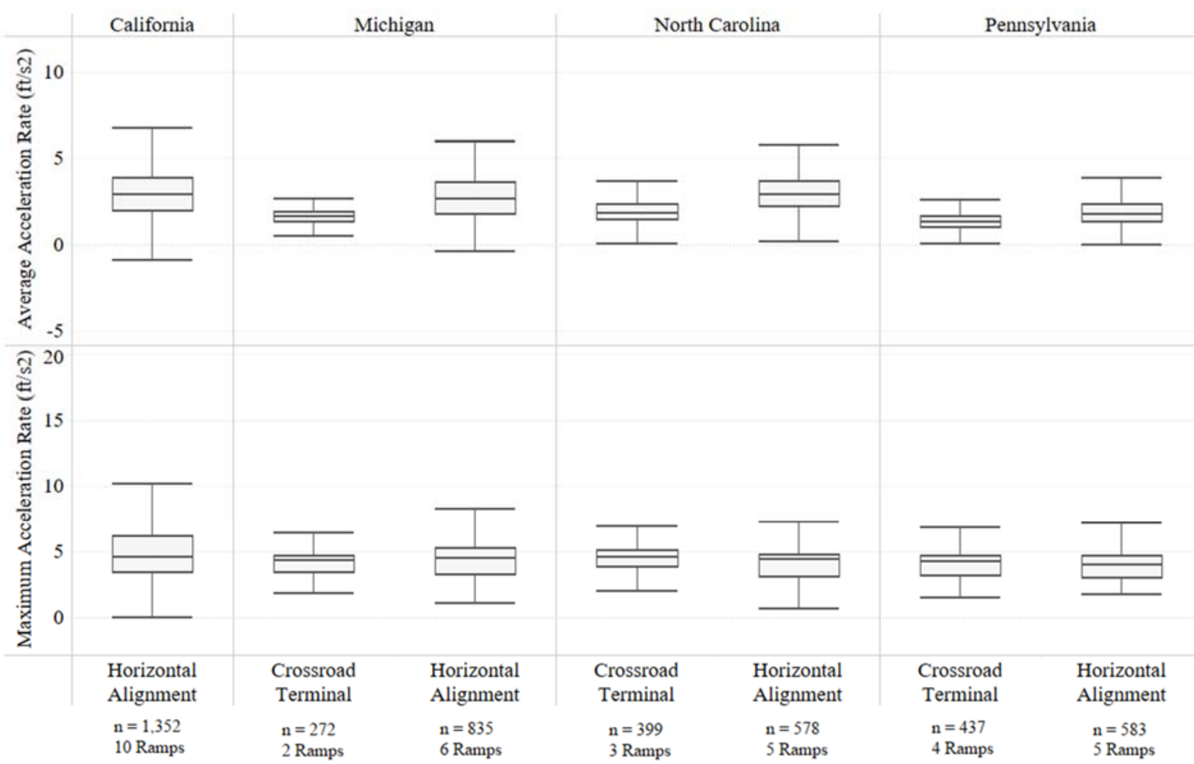


Figure 4-16. Acceleration Rate by Controlling Feature (Crossroad Terminal versus Curve).

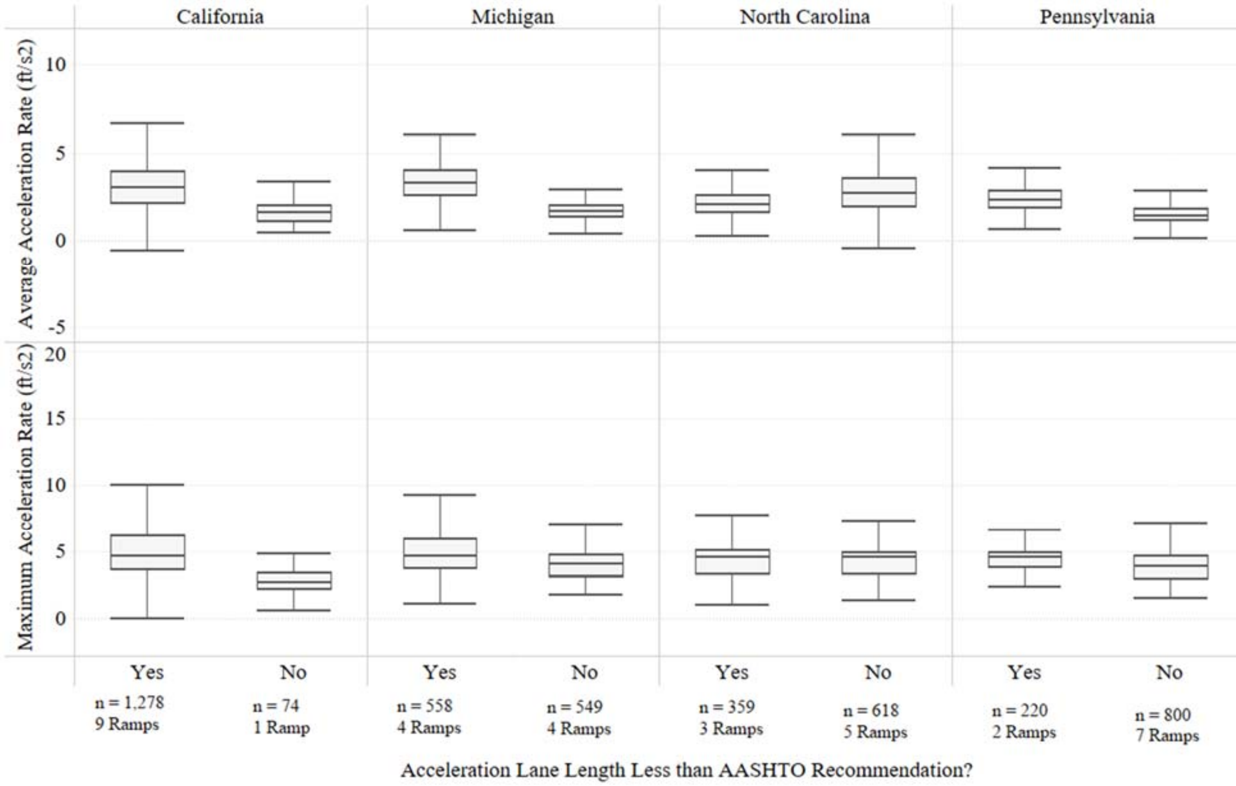


Figure 4-17. Acceleration Rate based on Minimum Criteria of Acceleration Lane Length from AASHTO Green Book (2018).

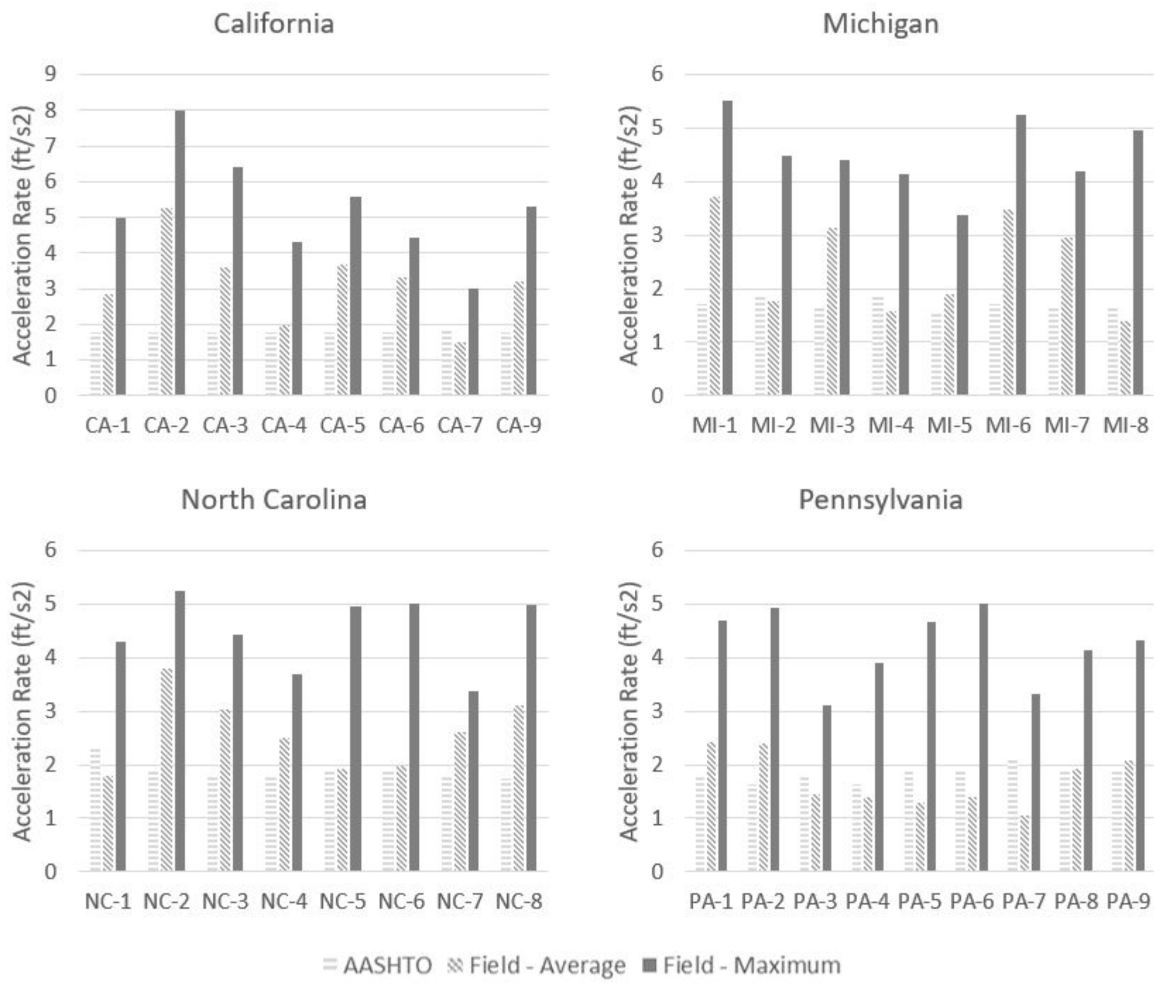


Figure 4-18. Acceleration Rate – Comparison between AASHTO-Assumed Values and Field Observations.

A.1.1.3 Results for Exit Ramps

This section provides the State-by-State plots of various design features for exit ramps based on the results from the field data collection.

A.1.1.3.1 Diverge Location

Figure 4-19 through Figure 4-22 display the distribution of diverge location based on the percentage of SCL used across Michigan, North Carolina, and Pennsylvania. The majority of the sites in California did not have SCLs, an example of which is shown in Figure 4-23. Hence, California sites were excluded from these series of figures. Table 4-11 displays the percentage of early diverge by site.

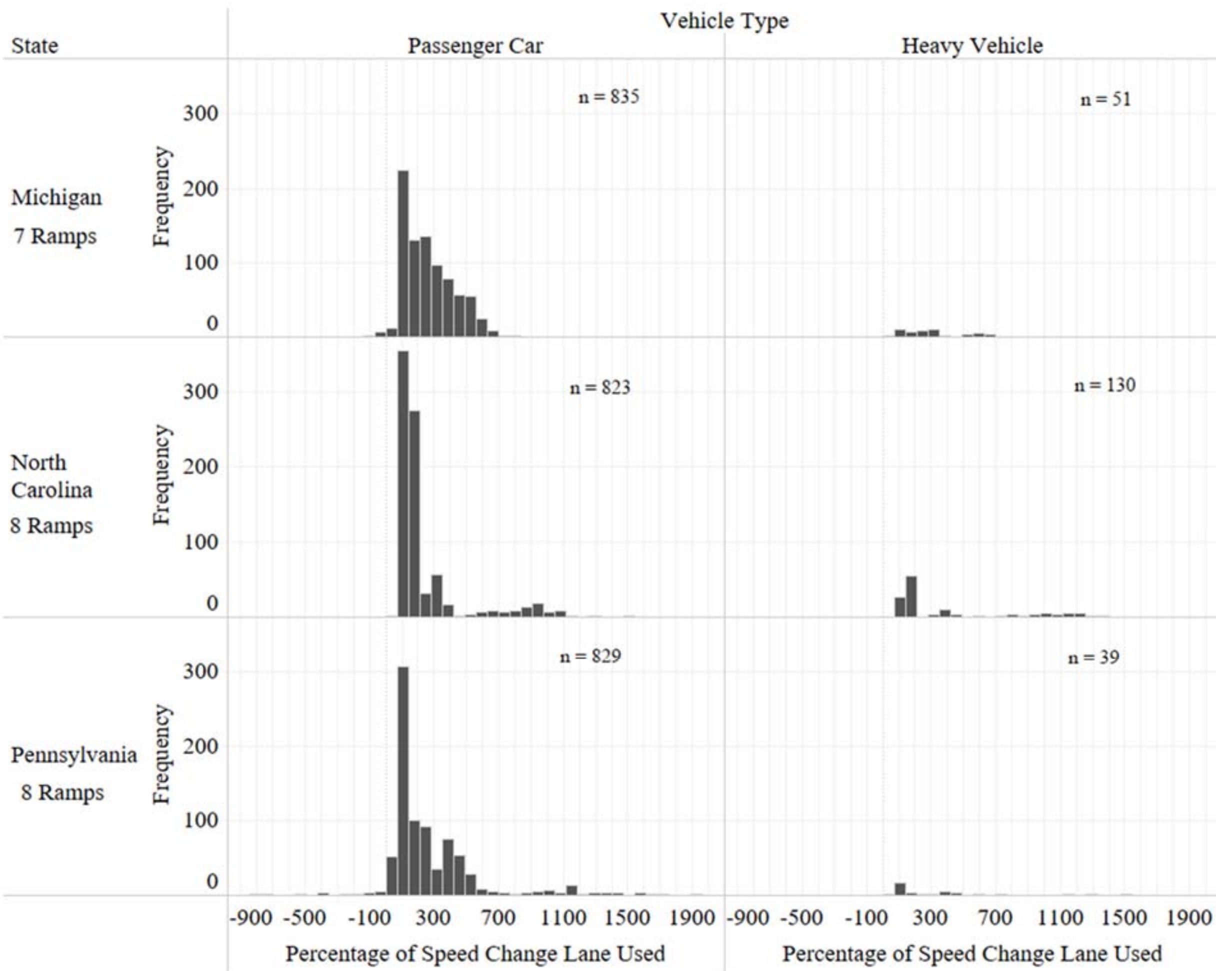


Figure 4-19. Percentage of SCL Used by Vehicle Type (Passenger Car versus Heavy Vehicle) for Exit Ramps.

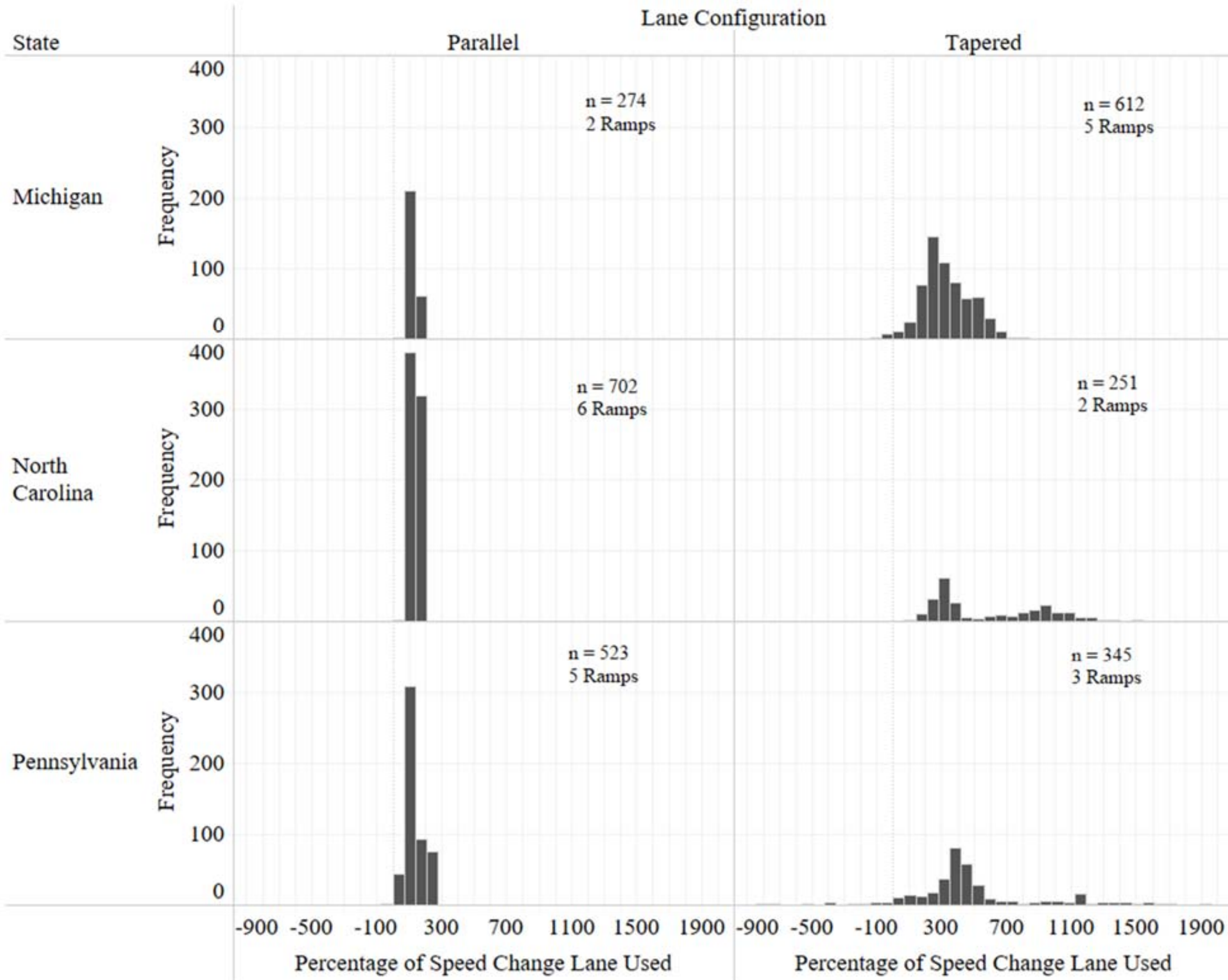


Figure 4-20. Percentage of SCL Used by Lane Configuration (Parallel versus Tapered) for Exit Ramps.

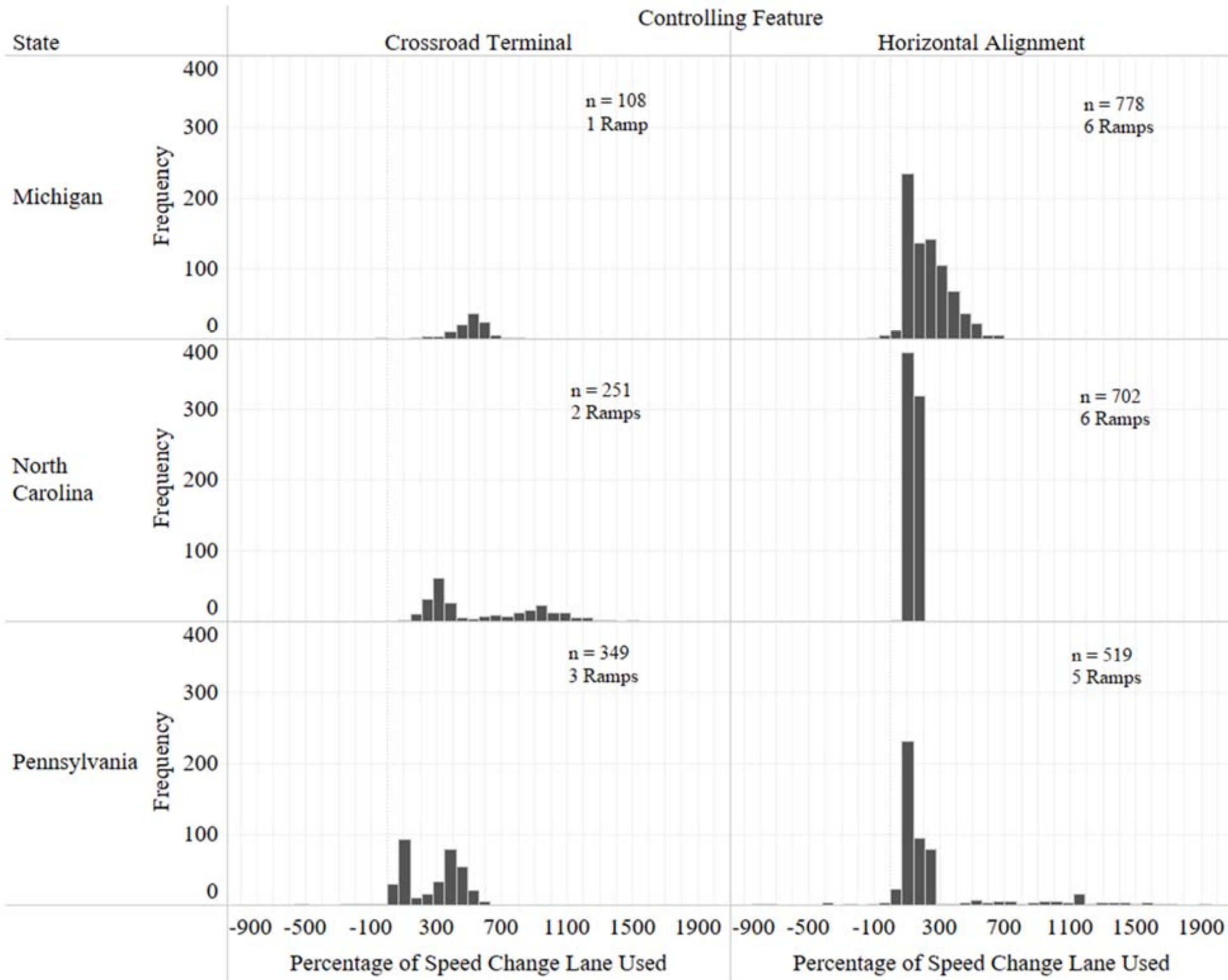


Figure 4-21. Percentage of SCL Used by Controlling Feature (Crossroad Terminal versus Curve) for Entrance Ramps.

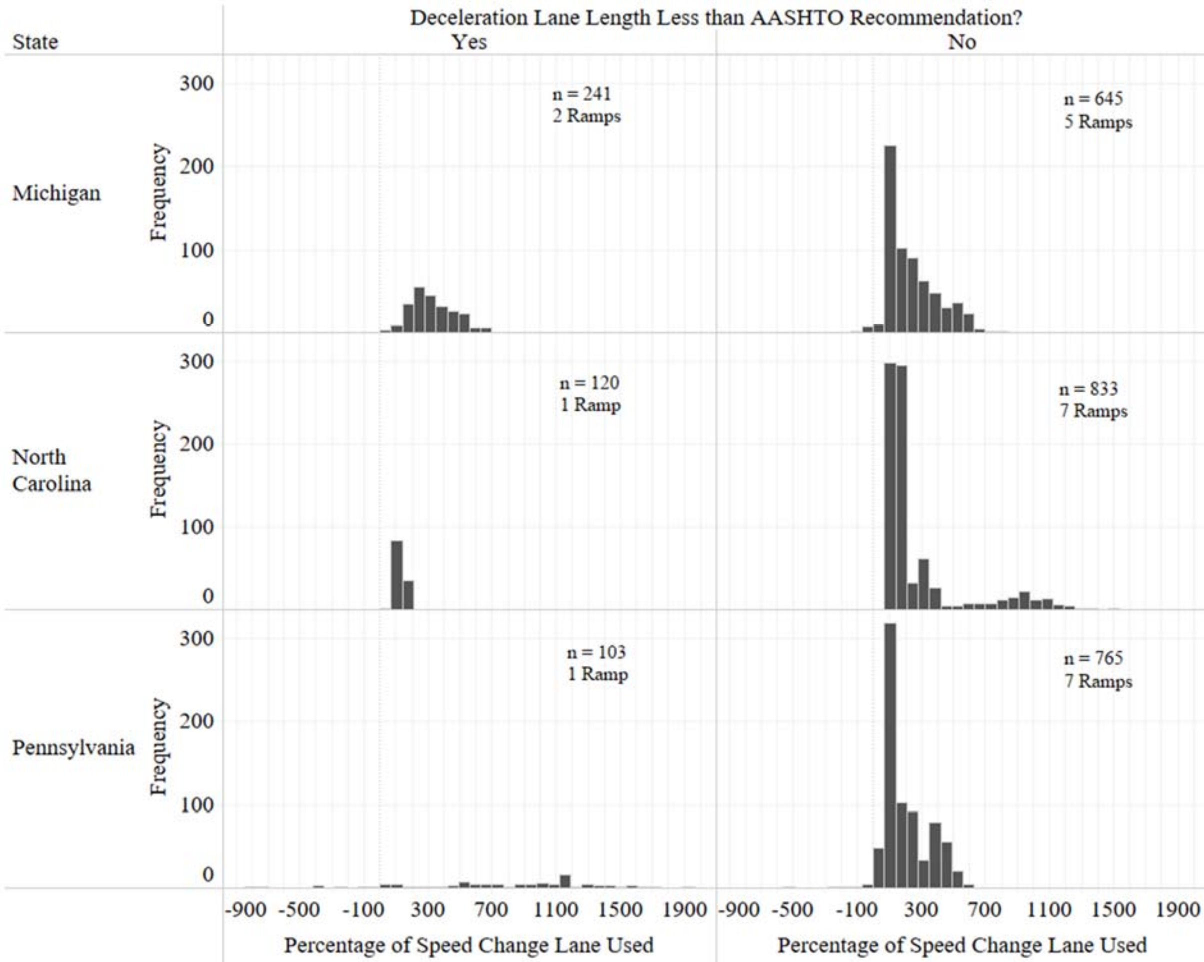


Figure 4-22. Percentage of SCL Used based on Minimum Criteria of Deceleration Lane Length from AASHTO Green Book (2018).

Figure 4-23. Example of California Site with No SCL (Image Credit: Google Earth™ Mapping Service).

Table 4-11. Percentage of Early Diverge.

Ramp ID	Diverge Type	SCL Length (ft)	Actual Deceleration Length (ft)	Green Book Minimum Deceleration Length (ft)	Percentage of Actual Length Less than Green Book Minimum Value	Number of Observations	Percentage of Early Diverge
CA-11	Tapered	0	791	440	80	155	61
CA-12	Tapered	0	335	500	-33	147	2
CA-13	Tapered	0	599	520	15	99	28
CA-14	Tapered	0	1,387	570	143	130	18
CA-15	Parallel	812	889	520	71	104	2
CA-16	Tapered	0	915	570	61	115	13
MI-9	Parallel	524	1,958	490	300	124	9
MI-10	Tapered	101	733	520	41	148	6
MI-11	Tapered	122	1,812	440	312	115	1
MI-12	Parallel	834	1,00	624	60	150	28
MI-13	Tapered	71	302	440	-31	130	2
MI-14	Tapered	0	200	520	-62	108	52
MI-15	Tapered	36	300	520	-42	111	36
MI-16	Tapered	53	910	615	48	108	9
NC-9	Parallel	313	404	342	18	118	19
NC-10	Parallel	406	410	423	-3	120	4
NC-11	Tapered	24	750	570	32	119	31
NC-12	Parallel	334	431	390	11	120	8
NC-13	Parallel	671	730	387	89	130	1
NC-14	Parallel	515	567	390	45	111	3
NC-15	Tapered	88	980	570	72	132	3
NC-16	Parallel	635	741	450	65	103	21
PA-10	Tapered	15	335	470	-29	103	1
PA-11	Parallel	145	740	459	61	08	0
PA-12	Parallel	135	855	396	116	105	00
PA-13	Parallel	462	1,826	615	197	107	0
PA-14	Tapered	21	1,385	480	189	122	0
PA-15	Parallel	549	1,220	396	208	99	0
PA-16	Tapered	37	1,870	576	225	120	0
PA-17	Parallel	455	964	380	154	104	0

A.1.1.3.2 Diverge Speed

This section presents the summary data of diverge speed differential. The speed differential was calculated based on the difference between the diverge speed (i.e., speed when the right wheels of the vehicle touch the line separating the mainline and ramp traffic) and mainline operating speed (i.e., speed of the mainline traffic on the rightmost lane under free-flow conditions). Figure 4-24 through Figure 4-29 show box plots of diverge speed differential by States. Figure 4-30 shows the diverge speed observed from the field data and the assumed values from the 2018 Green Book.

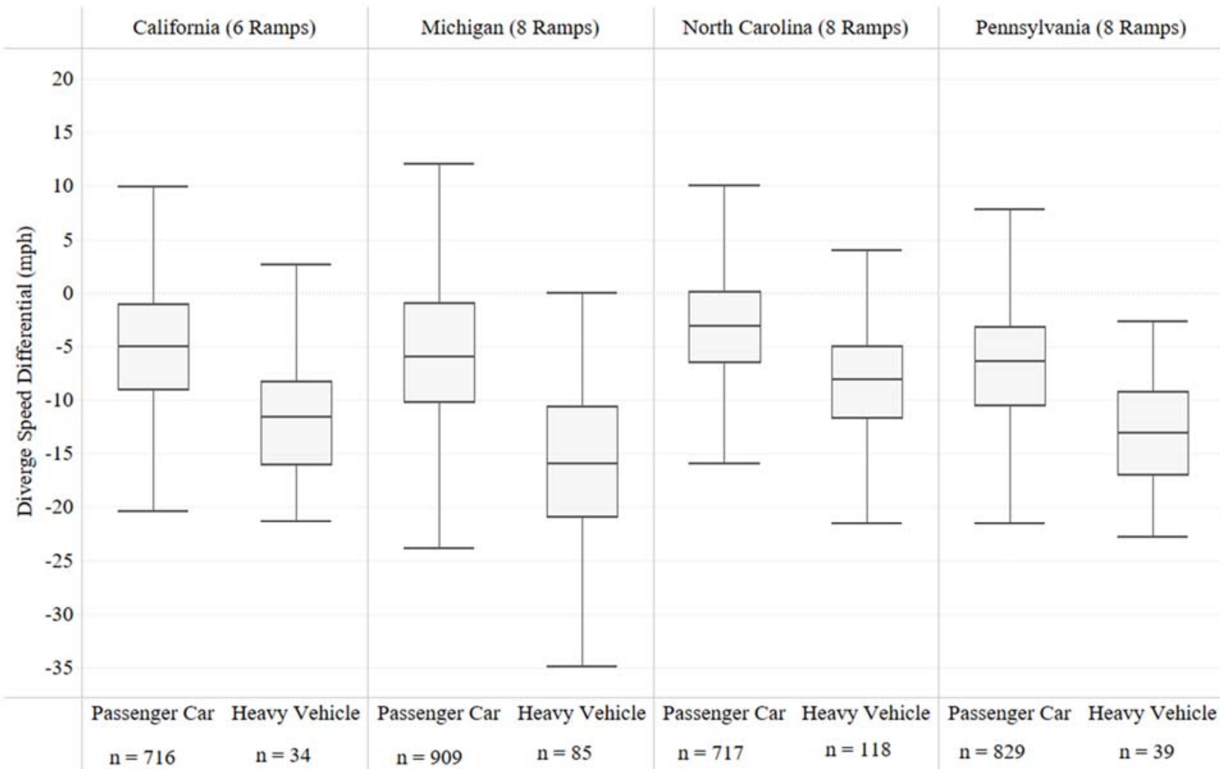


Figure 4-24. Diverge Speed Differential by Vehicle Type (Passenger Car versus Heavy Vehicle).

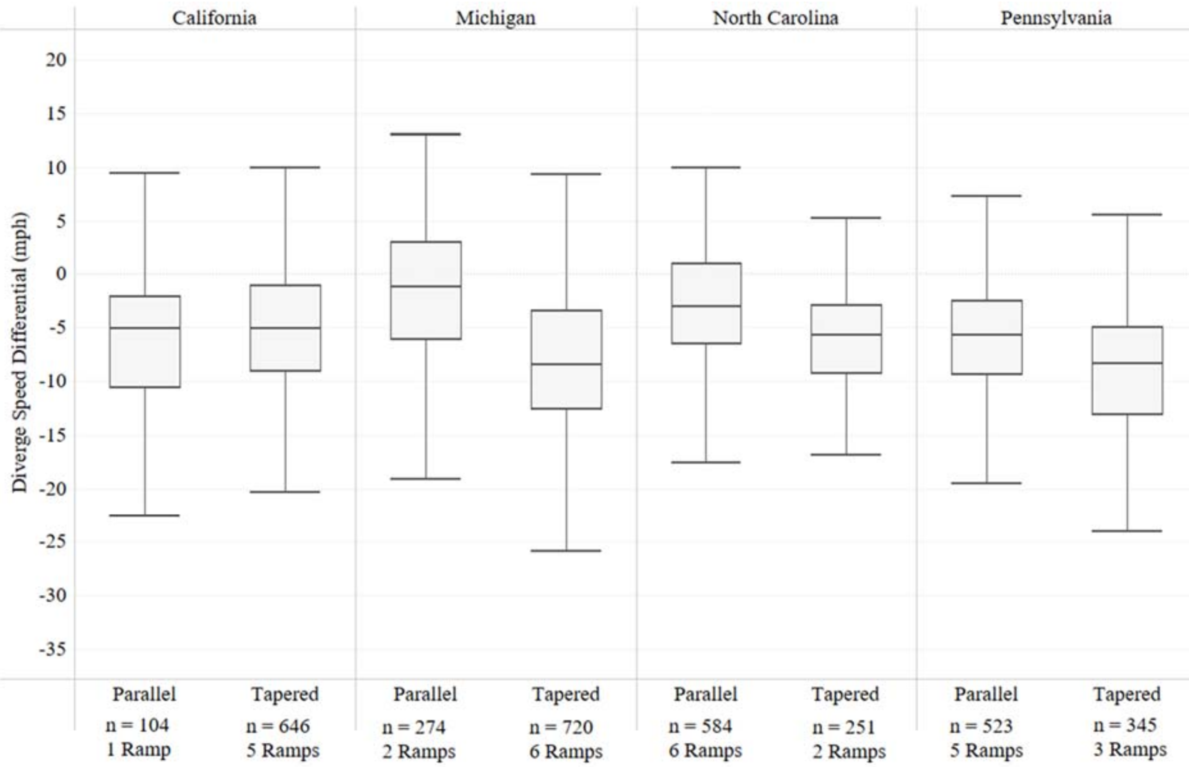


Figure 4-25. Diverge Speed Differential by Lane Configuration (Parallel versus Tapered).

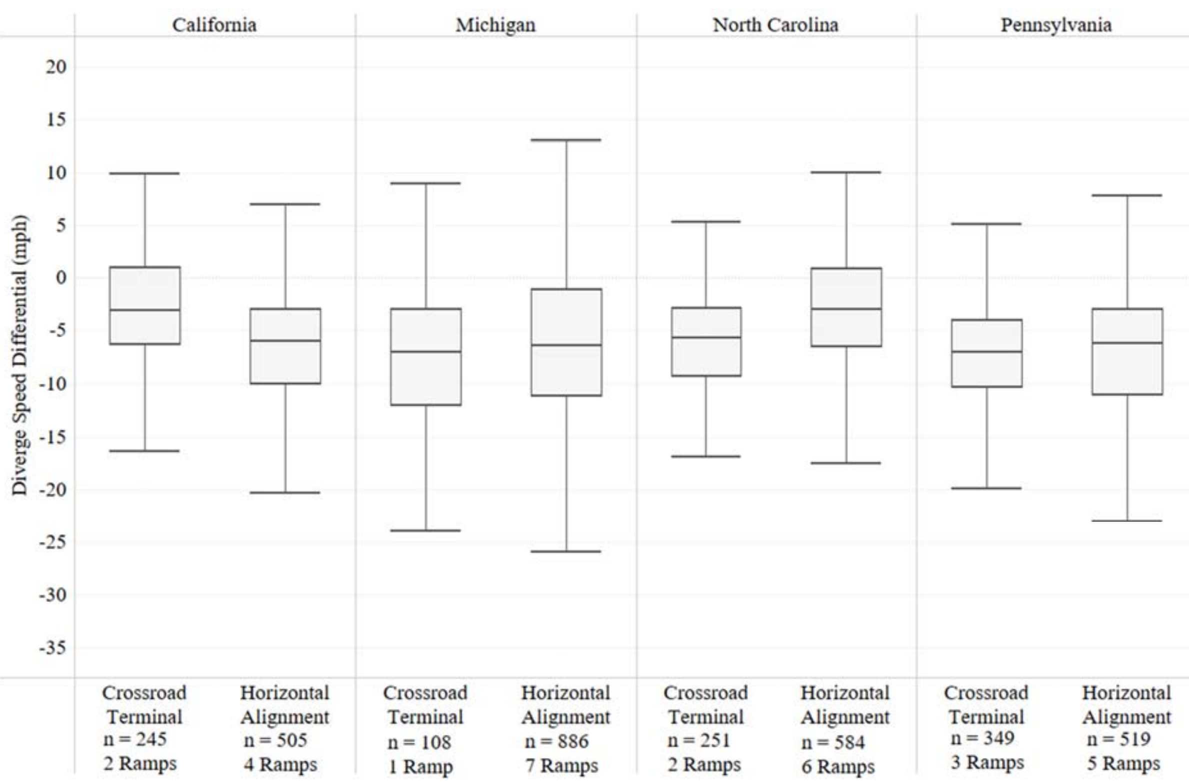


Figure 4-26. Diverge Speed Differential by Controlling Feature (Crossroad Terminal versus Curve).

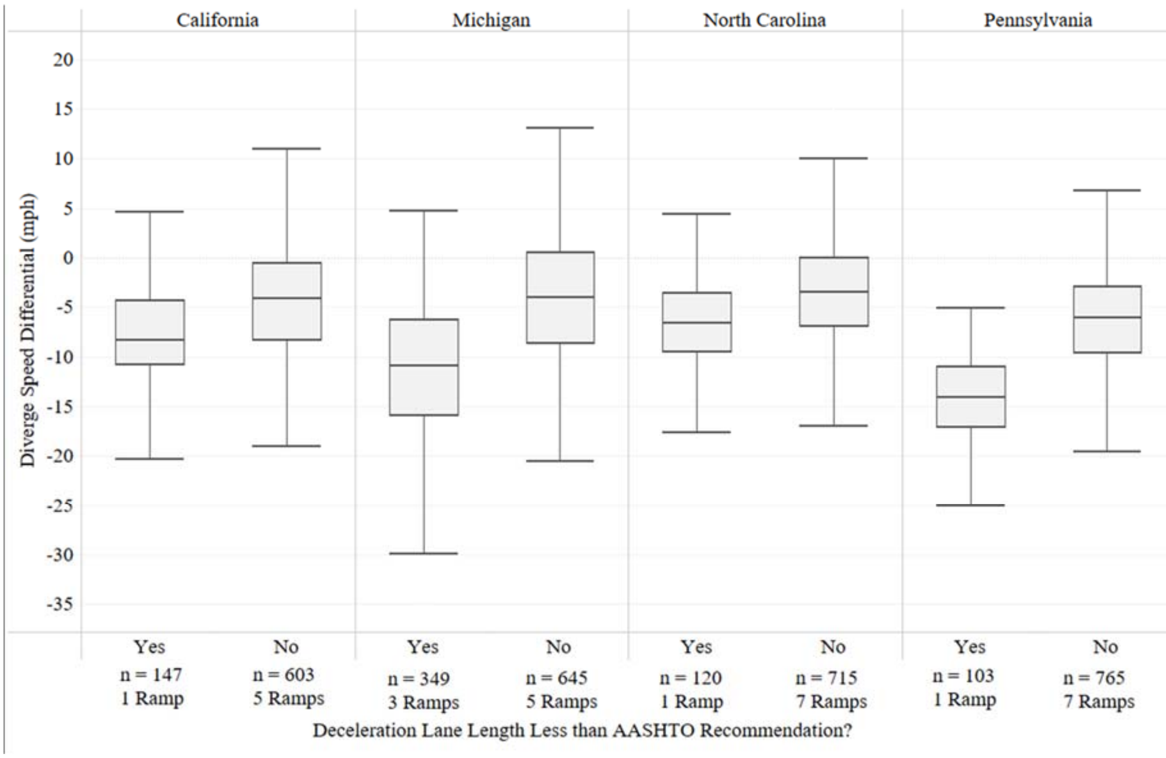


Figure 4-27. Diverge Speed Differential based on Minimum Criteria of Deceleration Lane Length from AASHTO Green Book (2018).

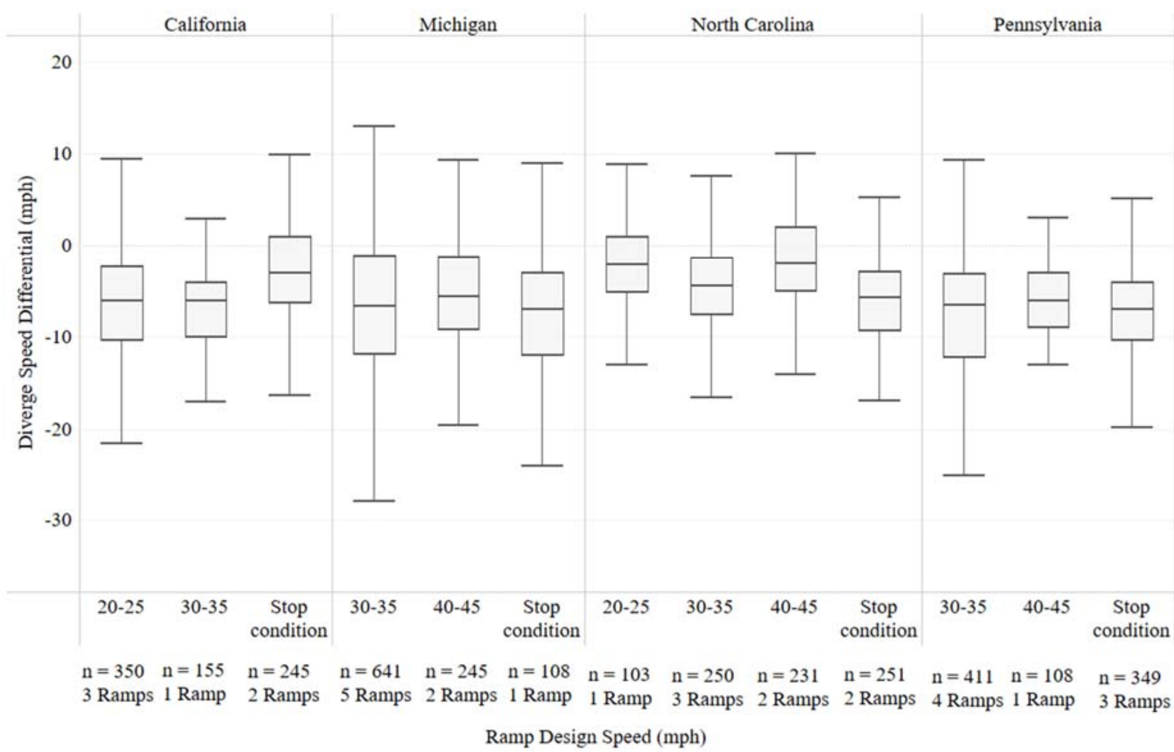


Figure 4-28. Diverge Speed Differential based on Ramp Design Speed.

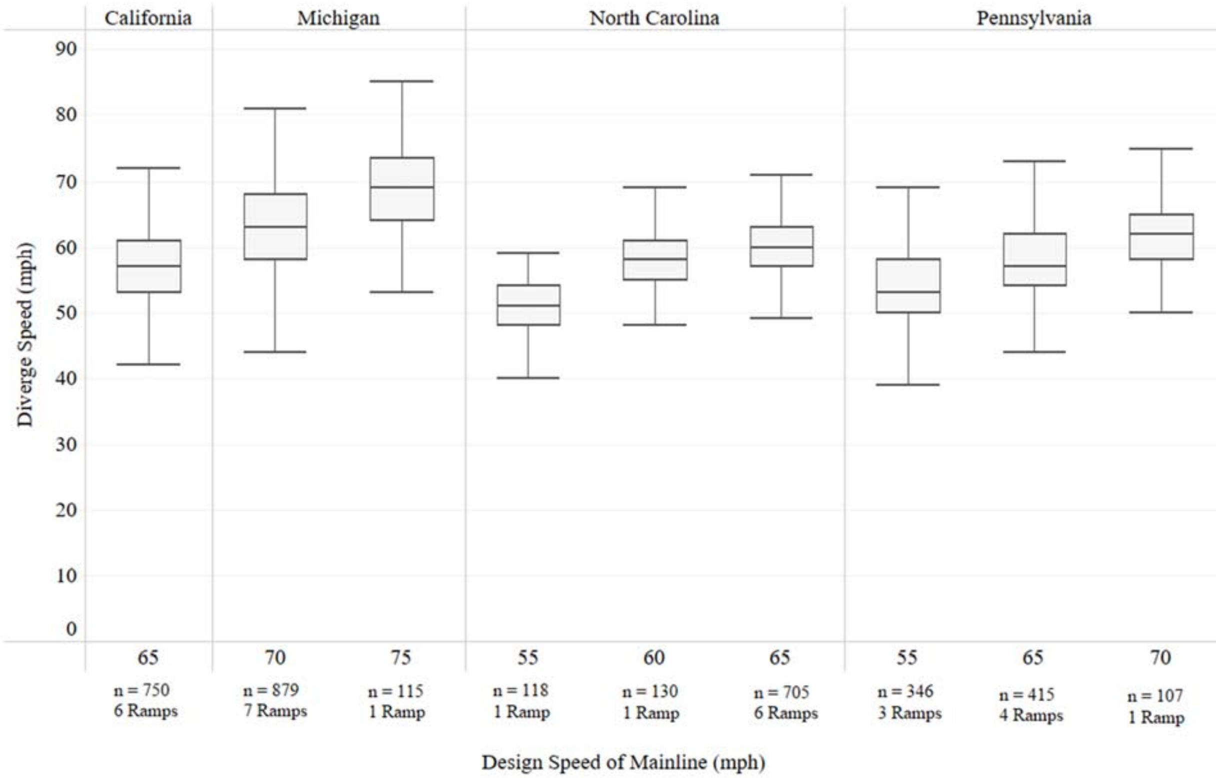


Figure 4-29. Diverge Speed based with Respect to Mainline Design Speed.

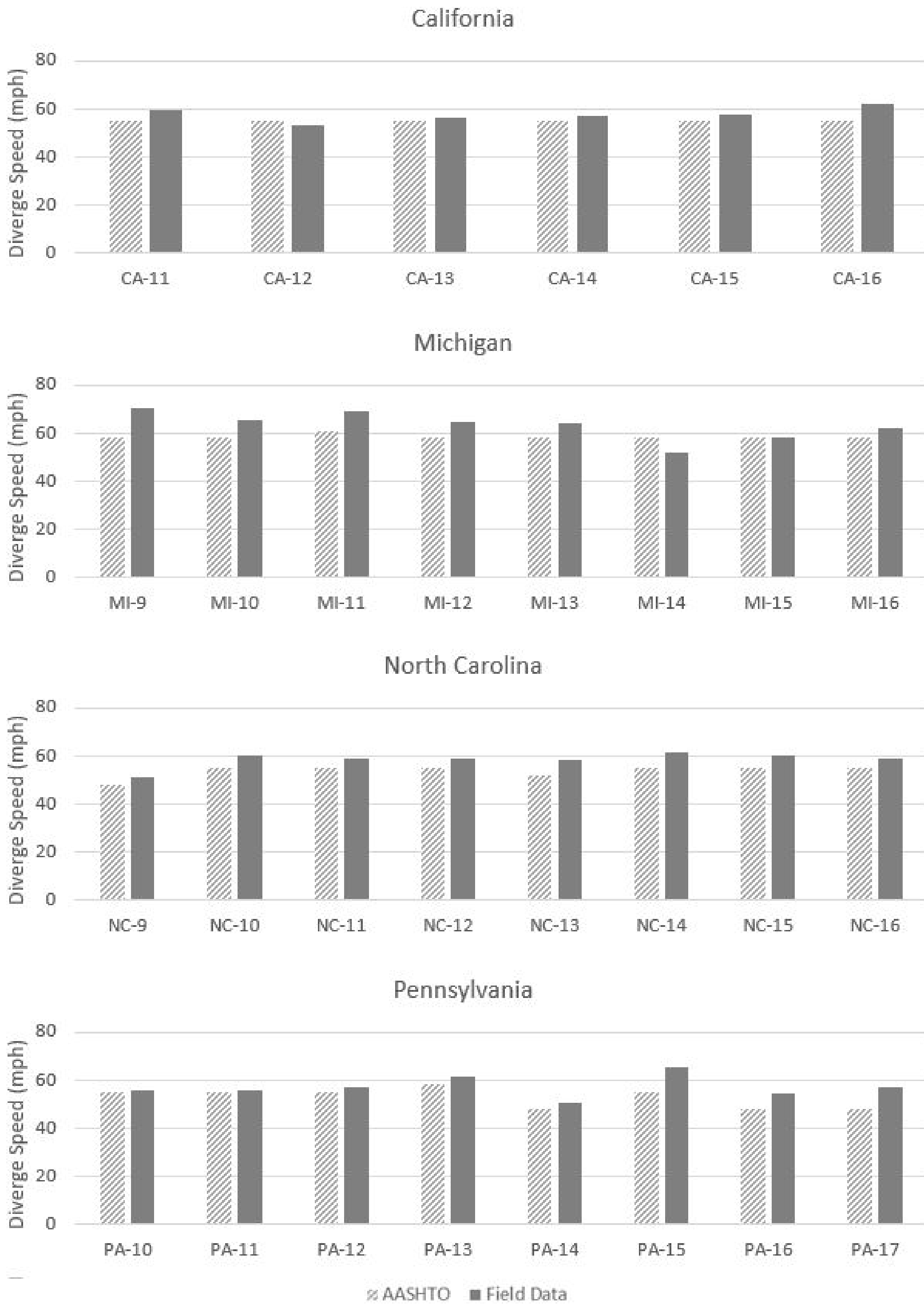


Figure 4-30. Diverge Speed – Comparison between AASHTO-Assumed Values and Field Observations.

A.1.1.3.3 Speed at Controlling Feature

Figure 4-31 and Figure 4-32 show the boxplots for speed differentials at the controlling feature (horizontal curve) by vehicle type and based on minimum values of deceleration lane length from the 2018 Green Book.

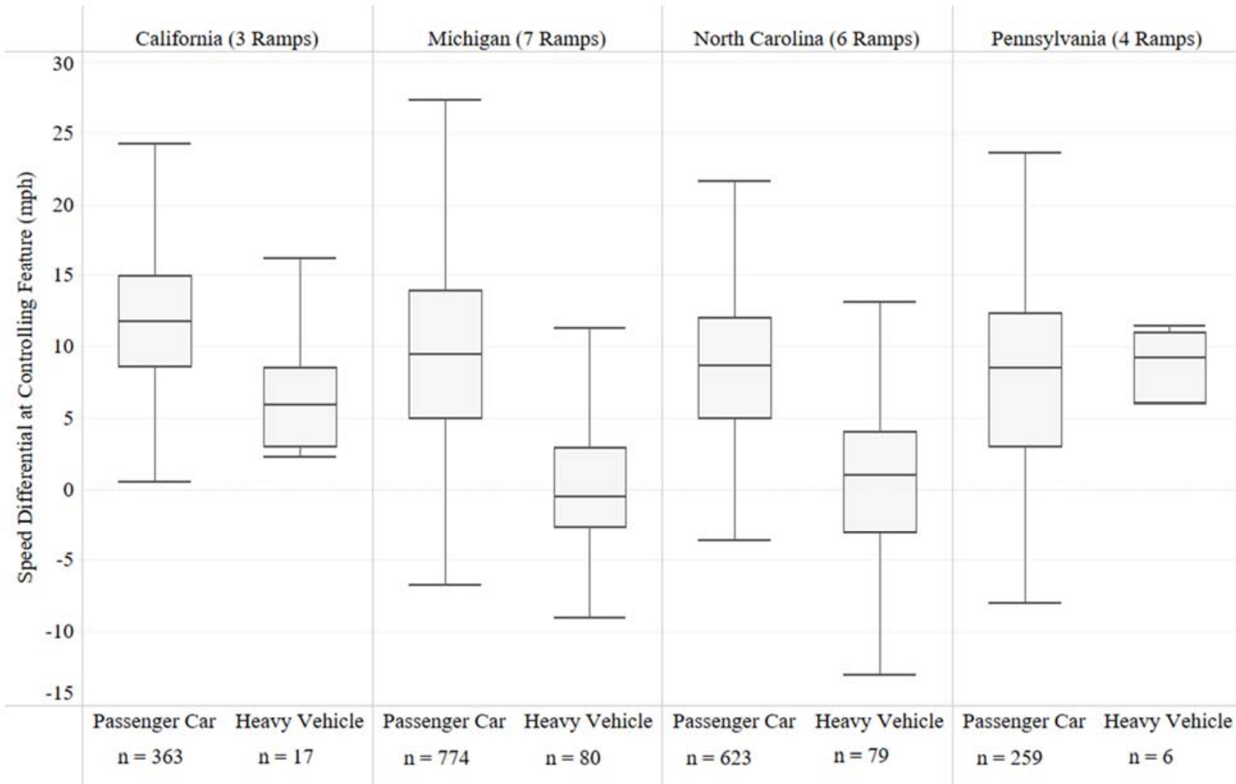


Figure 4-31. Speed Differential at Controlling Feature by Vehicle Type (Passenger Car versus Heavy Vehicle) for Exit Ramps.

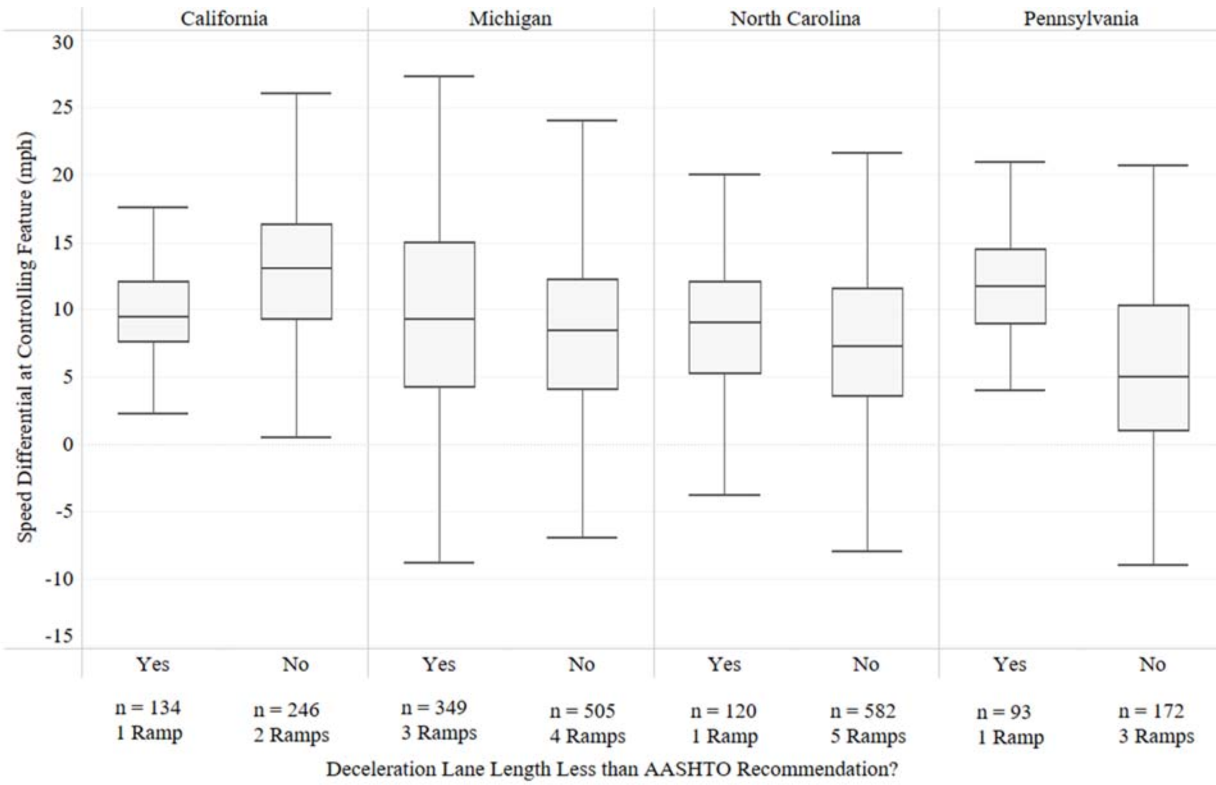


Figure 4-32***. Speed Differential at Controlling Feature based on Minimum Criteria of Deceleration Lane Length from AASHTO Green Book (2018).***

A.1.1.3.4 Deceleration Rate

Figure 4-33 through Figure 4-36 display deceleration rates based on different site characteristics and vehicle types.

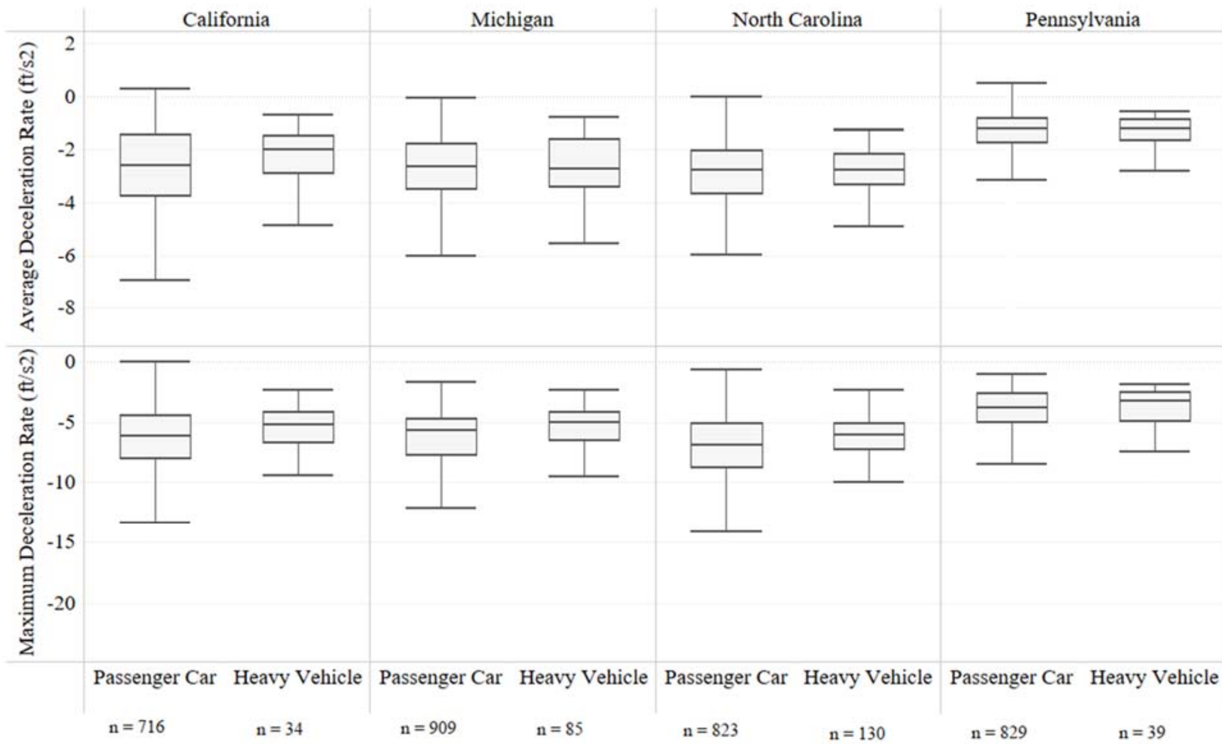


Figure 4-33. Deceleration Rate by Vehicle Type (Passenger Car versus Heavy Vehicle).

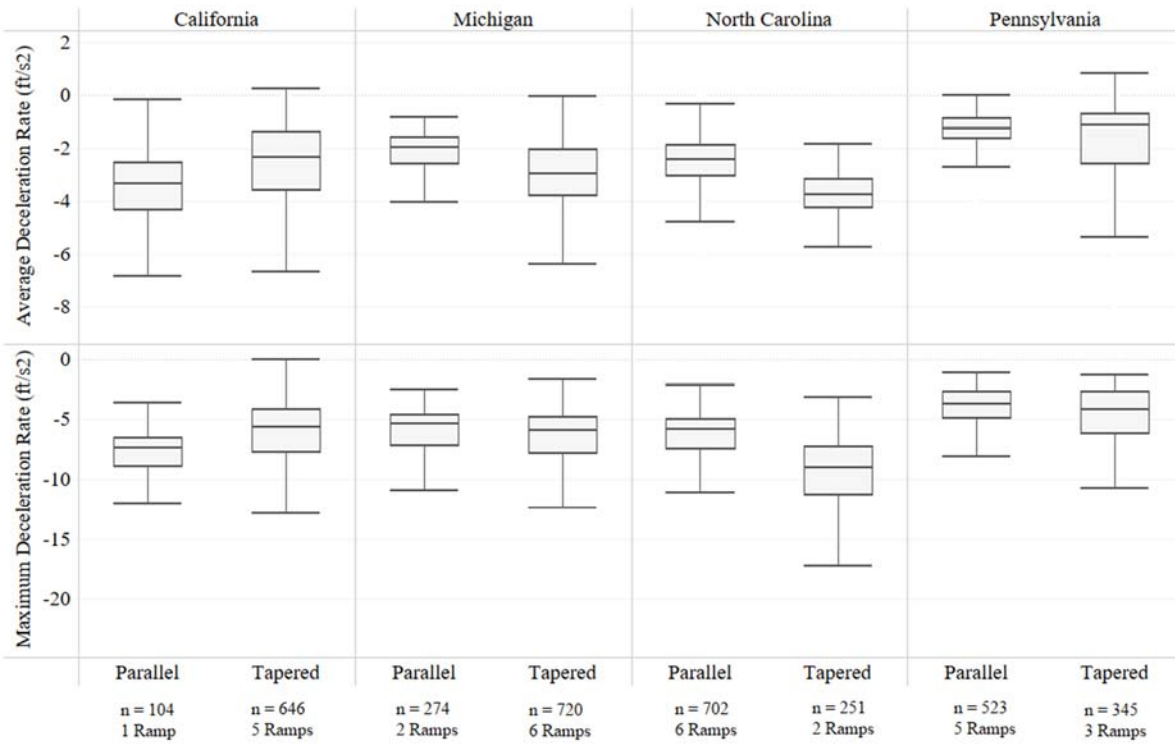


Figure 4-34. Deceleration Rate by Lane Configuration (Parallel versus Tapered).

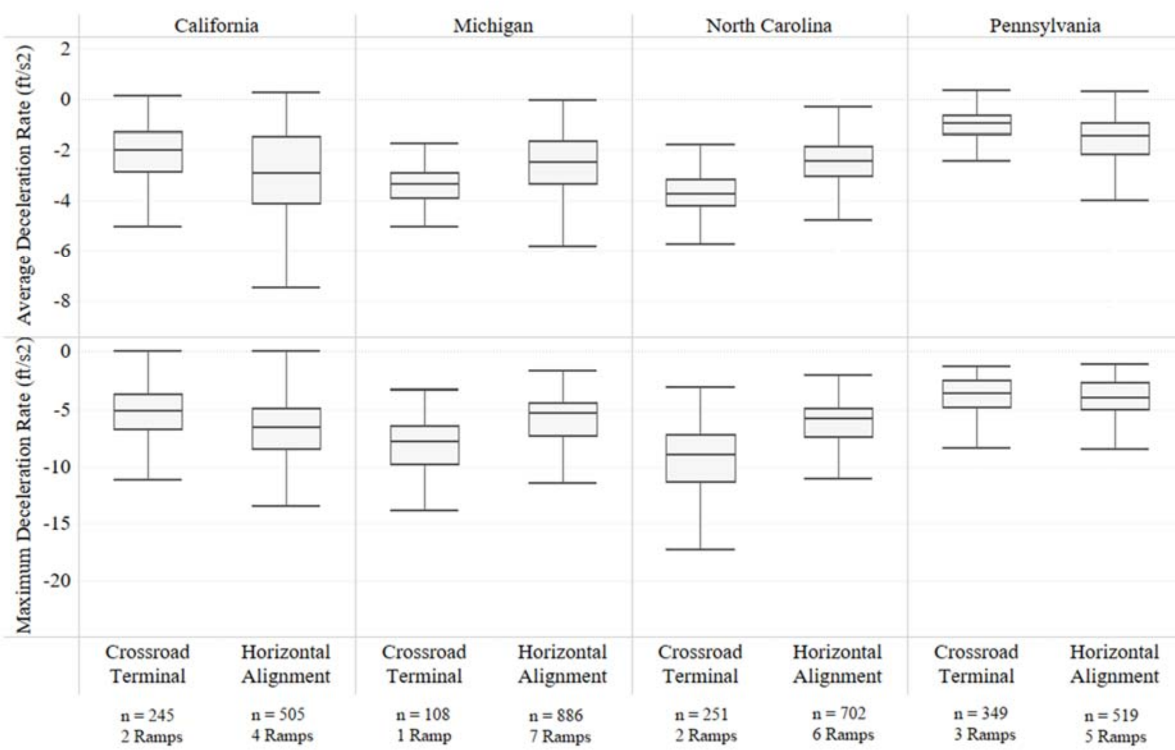


Figure 4-35. Deceleration Rate by Controlling Feature (Crossroad Terminal versus Curve).

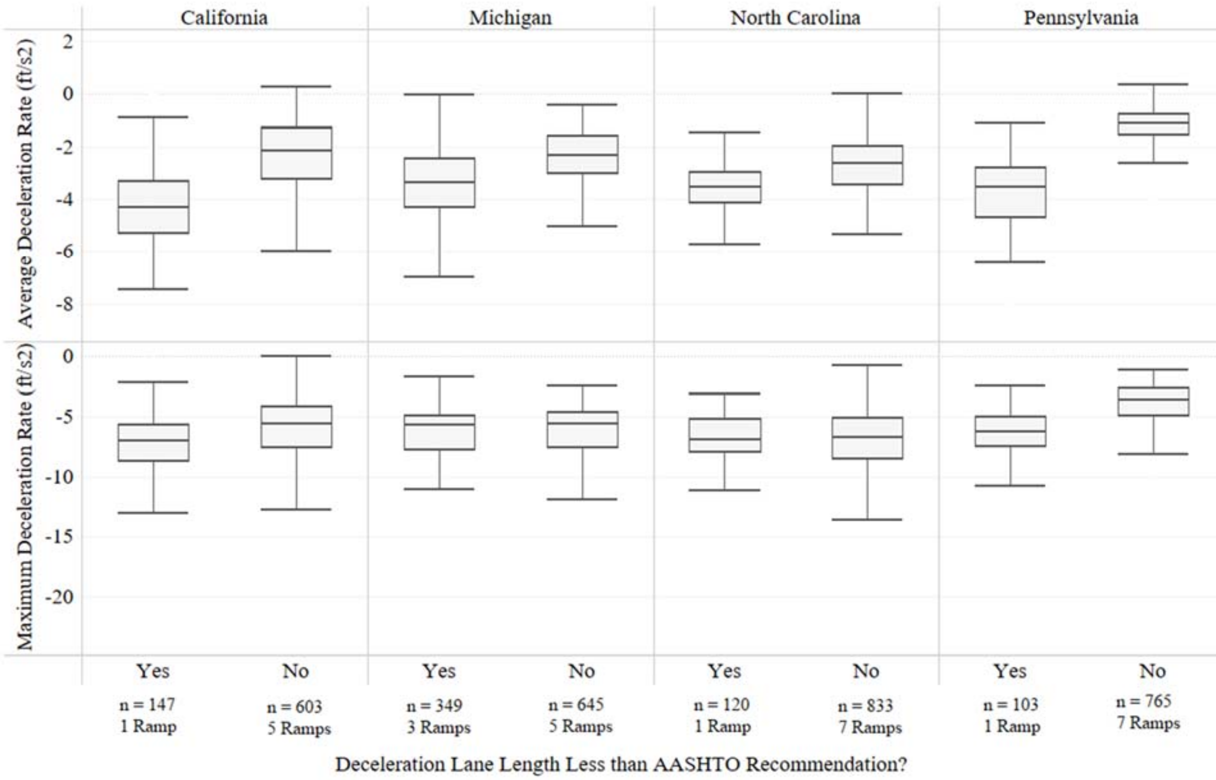


Figure 4-36. Deceleration Rate based on Minimum Criteria of Deceleration Lane Length from AASHTO Green Book (2018).

A.2 Acceleration Lane Length Design

A.2.1 Results

This section provides the results obtained from the probabilistic/reliability analysis of acceleration lane length design. Table 4-12 and Table 4-13 show the summary data of speed and acceleration parameters based on the field data collection, in addition to the assumed values from the 2018 Green Book. For analysis purposes, cases where the initial speed was not available from the field, the recommended values from the 2018 Green Book were assumed.

Figure 4-37, Figure 4-38, Figure 4-39, and Figure 4-40 show the summary of PNC results by site for Michigan, California, North Carolina, and Pennsylvania, respectively. The sites are sorted in ascending order based on the measured length of the acceleration lane.

Table 4-12. Comparison between 2018 Green Book and Field Data for Merge and Initial Speed.

State	Site	Freeway Design Speed (mph)	Ramp Design Speed (mph)	2018 Green Book		Field Data			
				Average Running Speed at CF (mph)	Merge Speed (mph)	Initial Speed (mph)		Merge Speed (mph)	
						μ	σ	μ	σ
California	CA-1	65	25	22	50	–	–	52.5	6.5
	CA-2	65	25	22	50	29.0	3.7	51.2	7.1
	CA-3	65	25	22	50	31.2	3.8	49.6	6.6
	CA-4	65	25	22	50	31.6	4.2	47.0	7.2
	CA-5	65	25	22	50	24.7	2.8	44.9	5.5
	CA-6	65	20	18	50	28.8	3.2	39.4	5.8
	CA-7	55	40	36	43	45.7	6.1	51.1	6.5
	CA-9	65	20	18	50	27.5	3.4	46.4	6.5
Michigan	MI-1	70	30	26	53	37.2	3.2	53.6	6.7
	MI-2	70	Stop	0	53	–	–	63.4	6.0
	MI-3	75	30	26	55	39.1	4.0	52.2	6.2
	MI-4	70	Stop	0	53	–	–	61.2	7.2
	MI-5	70	45	40	53	50.5	4.7	57.6	5.3
	MI-6	70	30	26	53	35.1	4.1	51.4	8.8
	MI-7	70	35	30	53	41.6	4.0	49.7	6.1
	MI-8	70	35	30	53	40.6	5.0	59.5	7.3
North Carolina	NC-1	50	Stop	0	39	–	–	46.3	5.8
	NC-2	50	35	30	39	36.0	3.7	45.7	5.7
	NC-3	70	25	22	53	34.4	3.6	46.0	8.7
	NC-4	65	30	26	50	34.1	4.1	43.4	7.5
	NC-5	65	Stop	0	50	–	–	53.8	7.4
	NC-6	65	Stop	0	50	–	–	56.2	7.6
	NC-7	65	30	26	50	34.7	4.3	41.9	7.3
	NC-8	65	35	30	50	31.0	4.2	44.7	7.5
Pennsylvania	PA-1	65	30	26	50	–	–	56.2	6.1
	PA-2	70	40	36	53	31.6	5.0	57.4	8.1
	PA-3	55	45	40	43	40.3	5.4	49.1	5.0
	PA-4	70	40	36	53	44.6	4.2	62.6	5.1
	PA-5	65	Stop	0	50	–	–	58.4	7.6
	PA-6	65	Stop	0	50	–	–	61.2	5.3
	PA-7	55	Stop	0	43	–	–	57.0	5.9
	PA-8	65	Stop	0	50	–	–	52.7	7.6
	PA-9	55	30	26	43	–	–	56.8	5.9

Note: – parameter is not available; Site CA-5 had data collected when ramp meter was active.

Table 4-13. Comparison between 2018 Green Book and Field Data for Acceleration Rates.

State	Site	Freeway Design Speed (mph)	Ramp Design Speed (mph)	2018 Green Book	Field Data			
				Acceleration Rate (ft/s^2)	Average Acceleration Rate (ft/s^2)		Maximum Acceleration Rate (ft/s^2)	
					μ	σ	μ	σ
California	CA-1	65	25	1.79	2.8	1.0	5.0	1.8
	CA-2	65	25	1.79	5.2	1.5	8.0	2.7
	CA-3	65	25	1.79	3.6	1.1	6.4	2.2
	CA-4	65	25	1.79	2.0	0.9	4.3	1.6
	CA-5	65	25	1.79	3.7	0.8	5.6	1.8
	CA-6	65	20	1.79	3.3	1.0	4.4	1.5
	CA-7	55	40	1.87	1.5	0.9	3.0	1.2
	CA-9	65	20	1.79	3.2	1.0	5.3	2.0
Michigan	MI-1	70	30	1.71	3.7	1.1	5.5	1.9
	MI-2	70	Stop	1.87	1.8	0.4	4.5	0.9
	MI-3	75	30	1.68	3.1	1.0	4.4	1.2
	MI-4	70	Stop	1.87	1.6	0.4	4.2	1.6
	MI-5	70	45	1.53	1.9	0.7	3.4	1.2
	MI-6	70	30	1.71	3.5	1.1	5.3	2.1
	MI-7	70	35	1.68	2.9	1.1	4.2	1.6
	MI-8	70	35	1.68	1.4	0.5	5.0	2.2
North Carolina	NC-1	50	Stop	2.28	1.8	0.8	4.3	1.3
	NC-2	50	35	2.03	3.8	1.1	5.3	2.0
	NC-3	70	25	1.77	3.0	1.3	4.4	2.1
	NC-4	65	30	1.76	2.5	1.1	3.7	1.4
	NC-5	65	Stop	1.92	1.9	0.6	5.0	1.8
	NC-6	65	Stop	1.92	2.0	0.7	5.0	1.6
	NC-7	65	30	1.76	2.6	0.8	3.4	1.2
	NC-8	65	35	1.73	3.1	1.1	5.0	1.7
Pennsylvania	PA-1	65	30	1.76	2.4	0.7	4.7	1.6
	PA-2	70	40	1.63	2.4	0.8	4.9	2.1
	PA-3	65	45	1.79	1.4	0.6	3.1	1.0
	PA-4	70	40	1.63	1.4	0.3	3.9	1.3
	PA-5	65	Stop	1.92	1.3	0.4	4.7	1.2
	PA-6	65	Stop	1.92	1.4	0.3	5.0	1.5
	PA-7	55	Stop	2.08	1.1	0.4	3.3	1.1
	PA-8	65	Stop	1.92	1.9	0.6	4.1	1.7
	PA-9	55	30	1.89	2.1	0.8	4.3	1.6

Note: Site CA-5 had data collected when ramp meter was active.

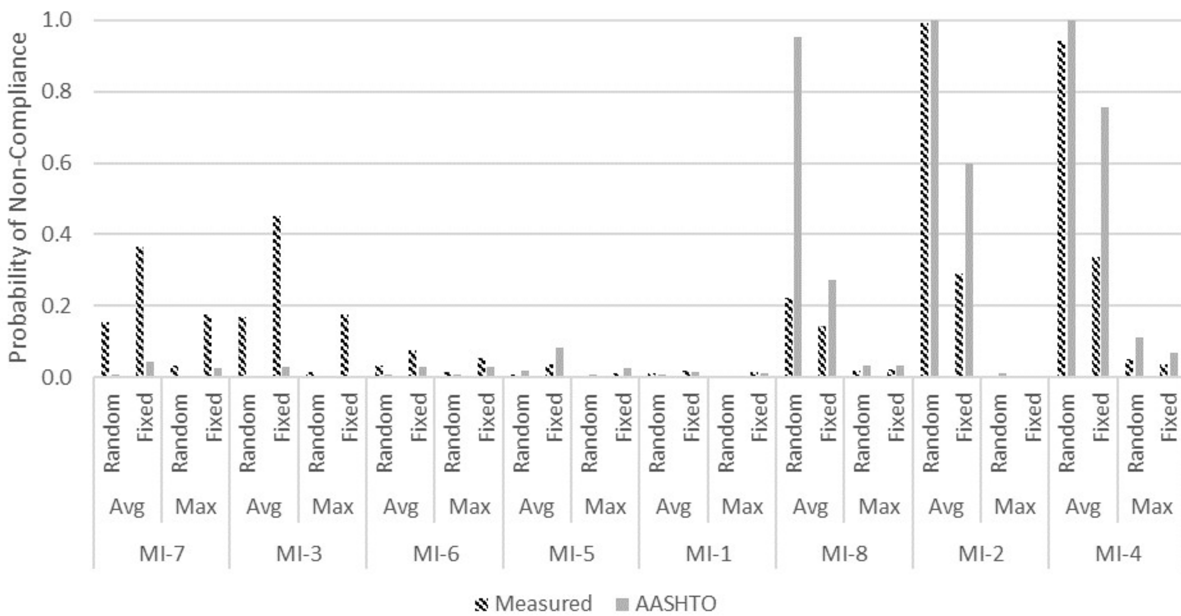


Figure 4-37. Probability of Non-Compliance by Site in Michigan.

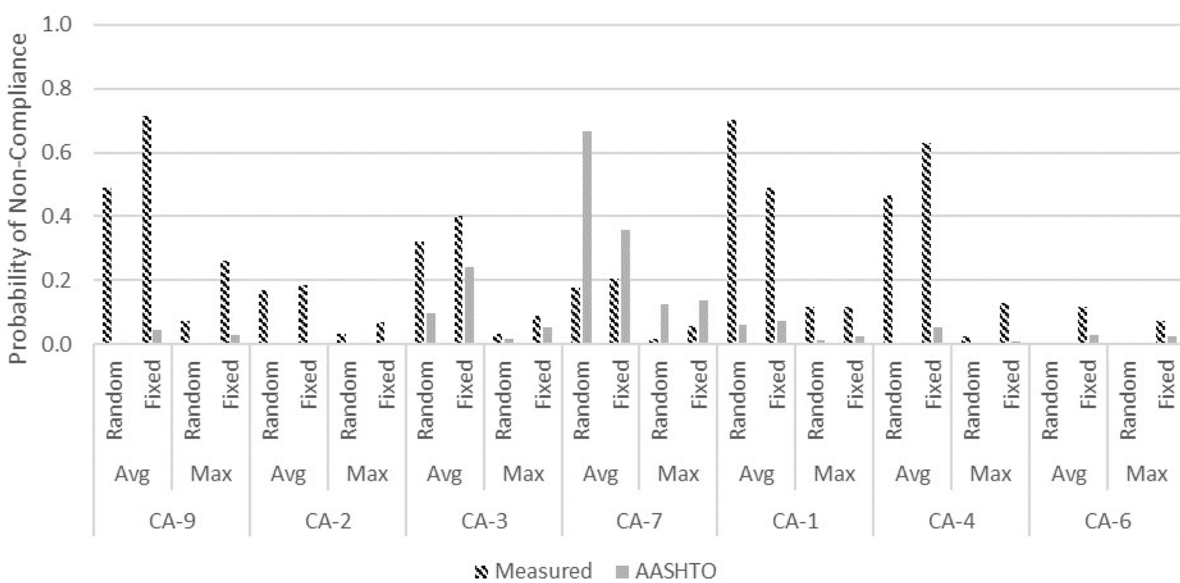


Figure 4-38. Probability of Non-Compliance by Site in California.

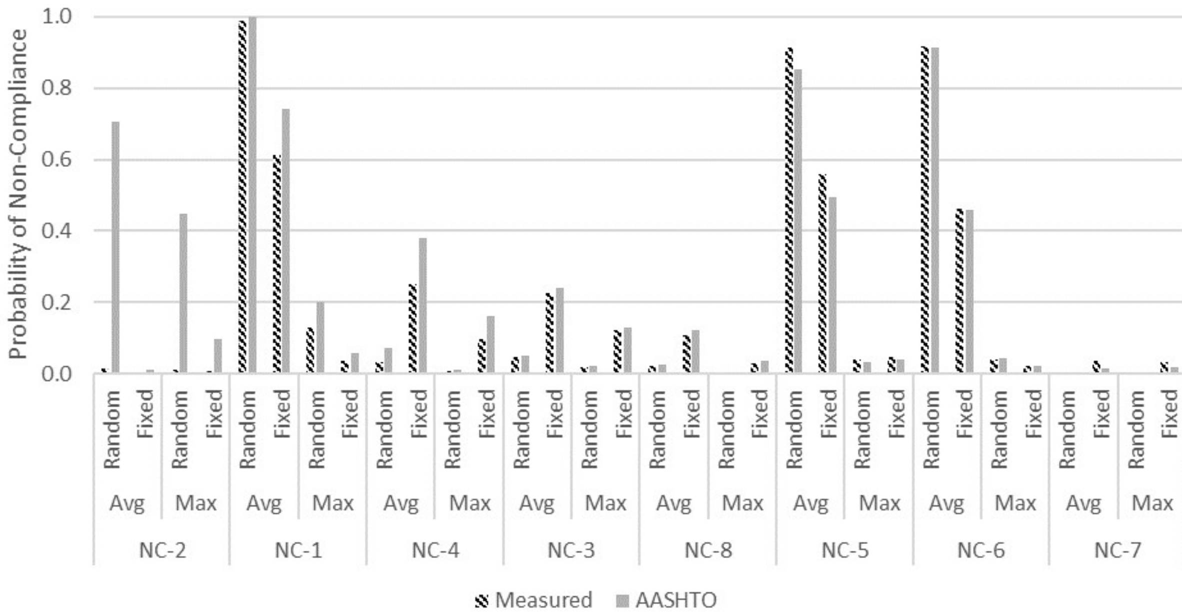


Figure 4-39. Probability of Non-Compliance by Site in North Carolina.

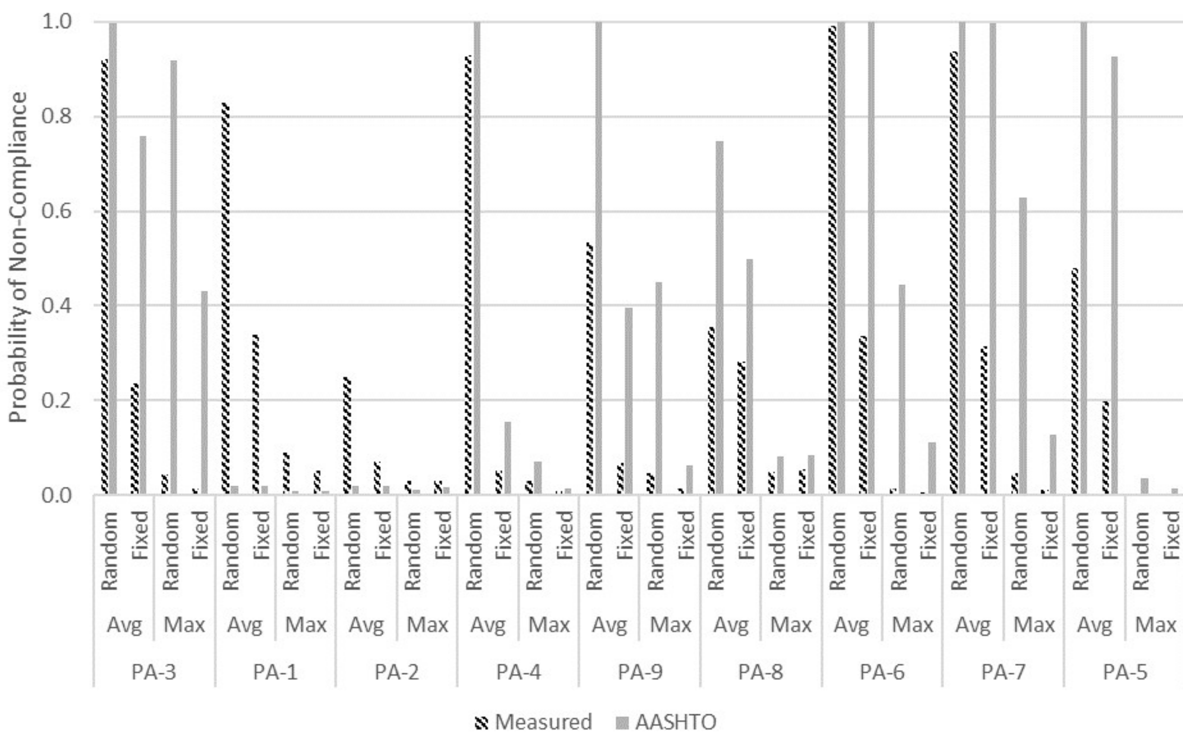


Figure 4-40. Probability of Non-Compliance by Site in Pennsylvania.

A.3 Deceleration Lane Length Design

A.3.1 Results

This section provides the results obtained from the probabilistic/reliability analysis of deceleration lane length design. Table 4-14 and Table 4-15 provide a comparison between the 2018 Green Book and field measurements on diverge speed and speed as the controlling feature, and deceleration rates, respectively. Sites where field speed data were not available, the speed was inferred to the average running speed from the 2018 Green Book based on the ramp design speed for the reliability-based analysis.

Figure 4-41, Figure 4-42, Figure 4-43, and Figure 4-44 depict the plots of PNC results by site for Michigan, California, North Carolina, and Pennsylvania, respectively. The sites are arranged in ascending order of site measurement of deceleration lane length.

Table 4-14. Comparison between 2018 Green Book and Field Data for Diverge Speed and Speed at Controlling Feature.

State	Site	Freeway Design Speed (mph)	Ramp Design Speed (mph)	2018 Green Book		Field Data			
				Average Running Speed at CF (mph)	Diverge Speed (mph)	Speed at CF (mph)		Diverge Speed (mph)	
						μ	σ	μ	σ
California	CA-11	65	35	30	55	49.8	5.2	59.3	5.1
	CA-12	65	25	22	55	34.8	3.3	53.0	4.9
	CA-13	65	20	18	55	29.7	3.9	56.2	5.7
	CA-14	65	Stop	0	55	–	–	57.0	6.0
	CA-15	65	20	18	55	–	–	57.4	6.9
	CA-16	65	Stop	0	55	–	–	61.8	5.6
Michigan	MI-9	70	35	30	58	41.4	5.2	70.5	5.9
	MI-10	70	30	26	58	42.5	4.6	65.2	5.5
	MI-11	75	45	40	61	47.9	5.4	69.1	6.3
	MI-12	70	30	26	58	38.6	4.7	64.6	5.3
	MI-13	70	40	36	58	56.0	5.2	63.7	5.1
	MI-14	70	30	26	58	35.6	6.4	51.7	6.1
	MI-15	70	30	26	58	36.6	4.6	58.3	5.7
	MI-16	70	Stop	0	58	–	–	61.8	6.0
North Carolina	NC-9	55	30	26	48	33.0	3.7	51.1	4.3
	NC-10	65	30	26	55	38.8	4.7	59.8	5.3
	NC-11	65	Stop	0	55	–	–	59.1	5.0
	NC-12	65	40	36	55	44.7	5.4	58.8	4.8
	NC-13	60	30	26	52	40.5	4.3	57.9	4.1
	NC-14	65	40	36	55	51.5	6.0	61.4	4.8
	NC-15	65	Stop	0	55	–	–	60.3	4.8
	NC-16	65	25	22	55	32.6	5.4	59.1	5.0
Pennsylvania	PA-10	65	30	26	55	41.9	4.5	55.7	4.9
	PA-11	65	45	40	55	47.3	4.7	55.3	4.0
	PA-12	65	35	30	55	45.2	3.9	57.2	5.0
	PA-13	70	Stop	0	58	–	–	61.6	5.1
	PA-14	55	Stop	0	48	–	–	50.3	5.4
	PA-15	65	35	30	55	–	–	65.2	4.9
	PA-16	55	Stop	0	48	–	–	54.6	4.7
	PA-17	55	30	26	48	–	–	56.6	6.6

Note: – parameter is not available.

Table 4-15. Comparison between 2018 Green Book and Field Data for Deceleration Rate.

State	Site	Freeway Design Speed (mph)	Ramp Design Speed (mph)	2018 Green Book		Field Data			
				Deceleration Rate (ft/s^2)		Average Deceleration Rate (ft/s2)		Maximum Deceleration Rate (ft/s^2)	
				Coasting	Braking	μ	σ	μ	σ
California	CA-11	65	35	-2.50	-7.49	-1.3	0.5	-4.5	2.4
	CA-12	65	25	-2.50	-8.03	-4.4	1.2	-7.7	2.2
	CA-13	65	20	-2.50	-7.27	-3.4	0.9	-8.3	2.4
	CA-14	65	Stop	-2.50	-7.55	-1.5	0.6	-4.3	1.5
	CA-15	65	20	-2.50	-7,27	-3.5	1.5	-7.9	2.3
	CA-16	65	Stop	-2.50	-7.55	-3.0	1.0	-7.2	3.0
Michigan	MI-9	70	35	-2.50	-7.60	-1.6	0.4	-4.9	2.0
	MI-10	70	30	-2.50	-7.53	-2.9	0.7	-7.2	2.5
	MI-11	75	45	-2.99	-7.62	-1.4	0.4	-4.0	1.4
	MI-12	70	30	-2.50	-7.53	-2.5	0.7	-6.5	2.1
	MI-13	70	40	-2.99	-7.45	-2.2	1.0	-4.7	1.9
	MI-14	70	30	-2.50	-7.53	-3.7	1.1	-6.6	2.4
	MI-15	70	30	-2.50	-7.53	-4.5	1.1	-8.0	2.5
	MI-16	70	Stop	-2.50	-7.40	-3.4	0.7	-8.3	2.8
North Carolina	NC-9	55	30	-2.01	-7.86	-3.0	0.9	-6.7	2.3
	NC-10	65	30	-2.50	-7.76	-3.6	1.0	-7.2	2.8
	NC-11	65	Stop	-2.50	-7.55	-4.0	0.8	-9.9	3.3
	NC-12	65	40	-2.50	-7.51	-2.3	0.7	-5.6	1.9
	NC-13	60	30	-2.98	-7.70	-1.9	0.6	-6.4	2.1
	NC-14	65	40	-2.50	-7.51	-1.8	0.7	-4.5	1.3
	NC-15	65	Stop	-2.50	-7.55	-3.5	0.7	-9.1	2.8
	NC-16	65	25	-2.50	-8.03	-2.6	0.6	-7.7	2.4
Pennsylvania	PA-10	65	30	-2.50	-7.76	-3.8	1.3	-6.5	2.1
	PA-11	65	45	-2.50	-7.66	-1.1	0.6	-3.7	1.3
	PA-12	65	35	-2.50	-7.49	-1.4	0.6	-4.1	1.2
	PA-13	70	Stop	-2.50	-7.40	-1.3	0.4	-4.1	1.4
	PA-14	55	Stop	-2.01	-7.10	-0.8	0.6	-4.2	2.3
	PA-15	65	35	-2.50	-7.49	-1.2	0.5	-3.7	1.2
	PA-16	55	Stop	-2.01	-7.10	-0.9	0.5	-3.3	1.4
	PA-17	55	30	-2.01	-7.86	-1.6	0.8	-3.7	1.1

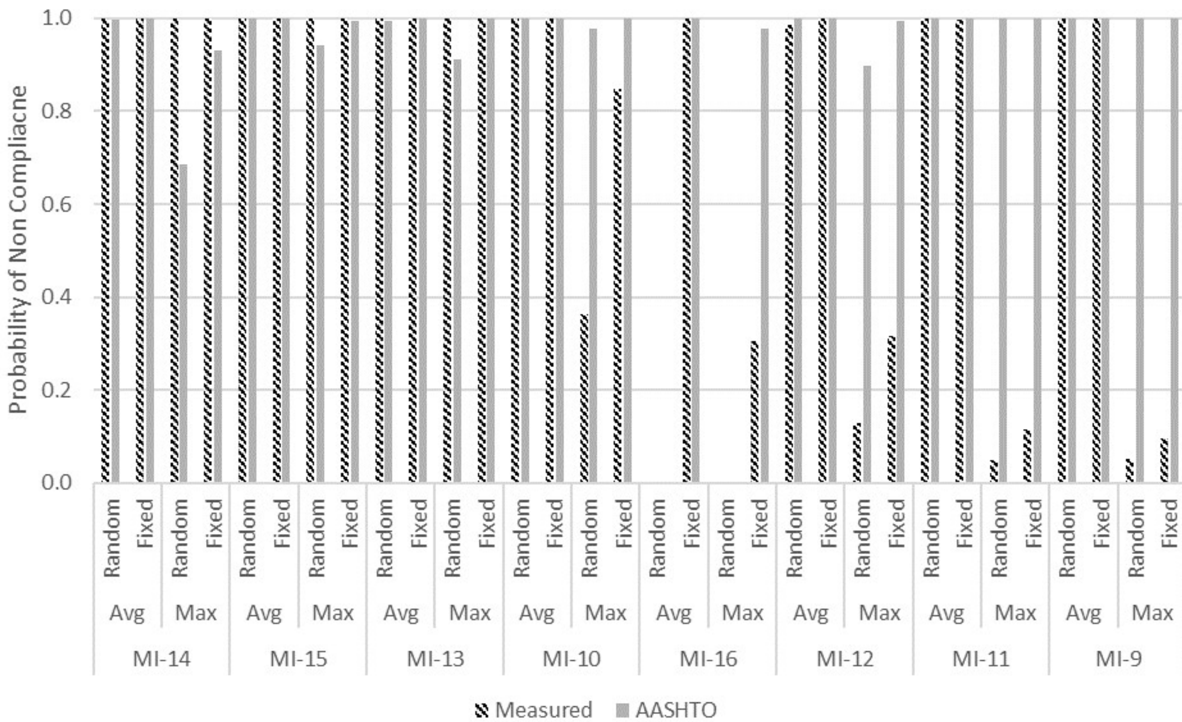


Figure 4-41. PNC for Michigan.

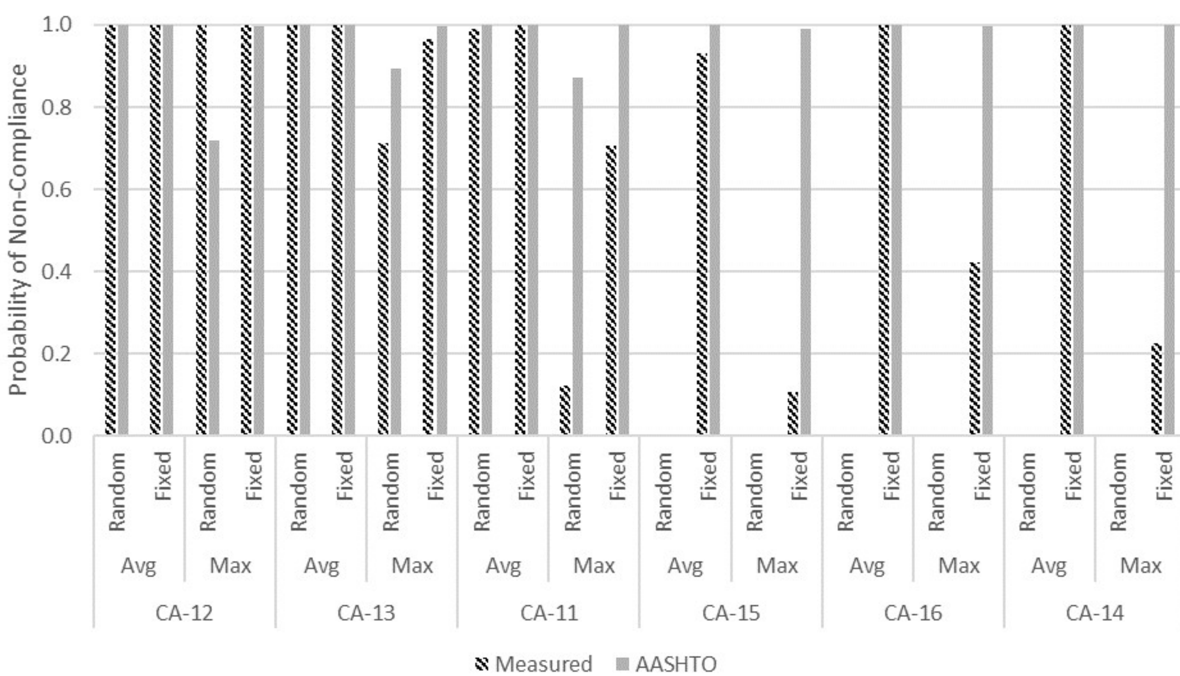


Figure 4-42. PNC for California.

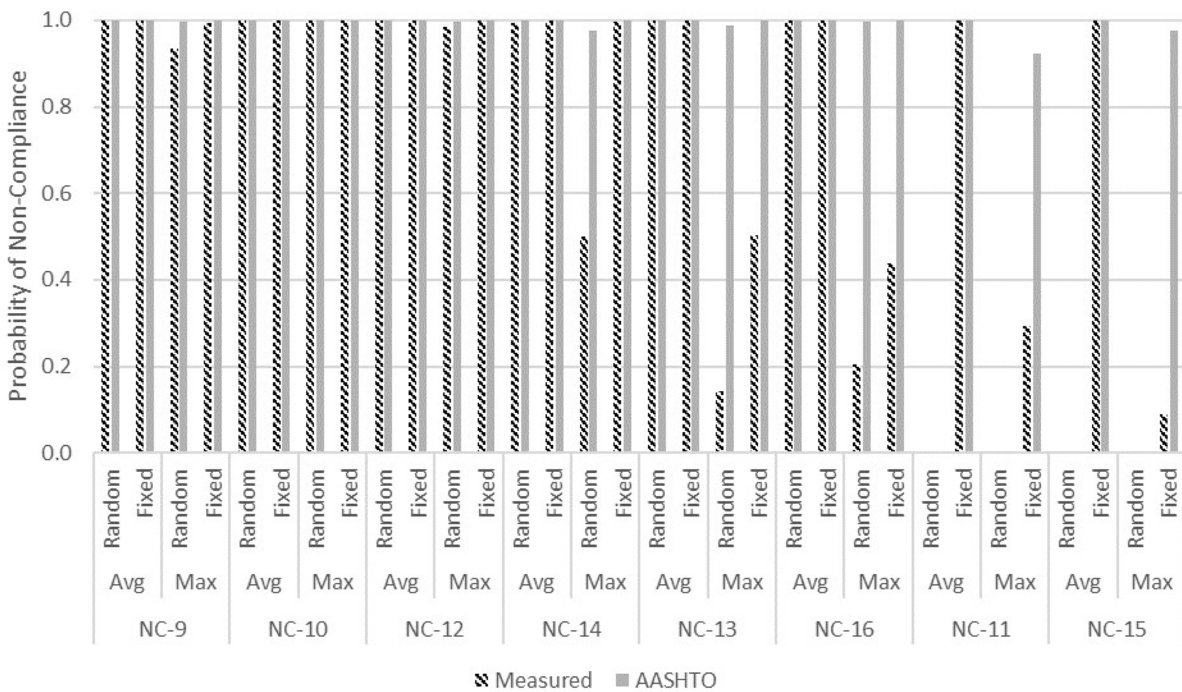


Figure 4-43. PNC for North Carolina.

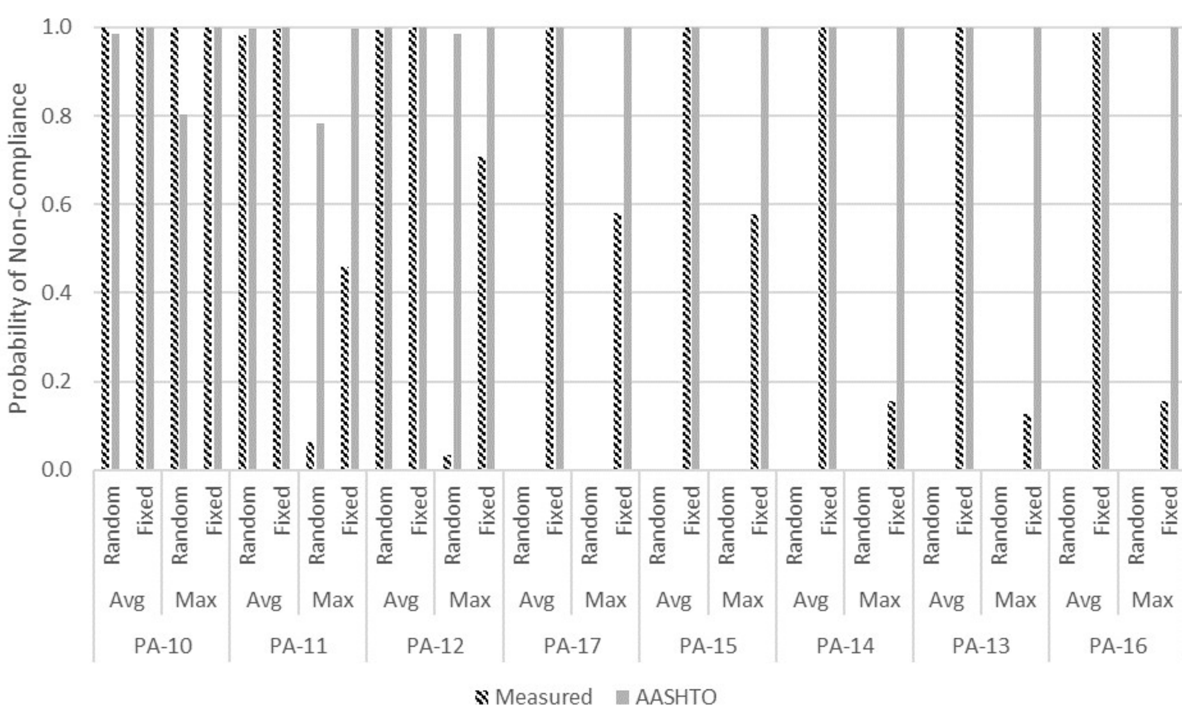


Figure 4-44. PNC for Pennsylvania.

Appendix B Reliability-Based Analysis for Entrance Ramps

This section presents the frequency distributions of speed and acceleration parameters utilized in the reliability-based analyses for entrance ramps. The frequency distribution of demand acceleration lane lengths based on maximum and average acceleration rates, and its comparison with the field measured acceleration lane length and AASHTO Green Book-recommended values are also presented separately for each site.

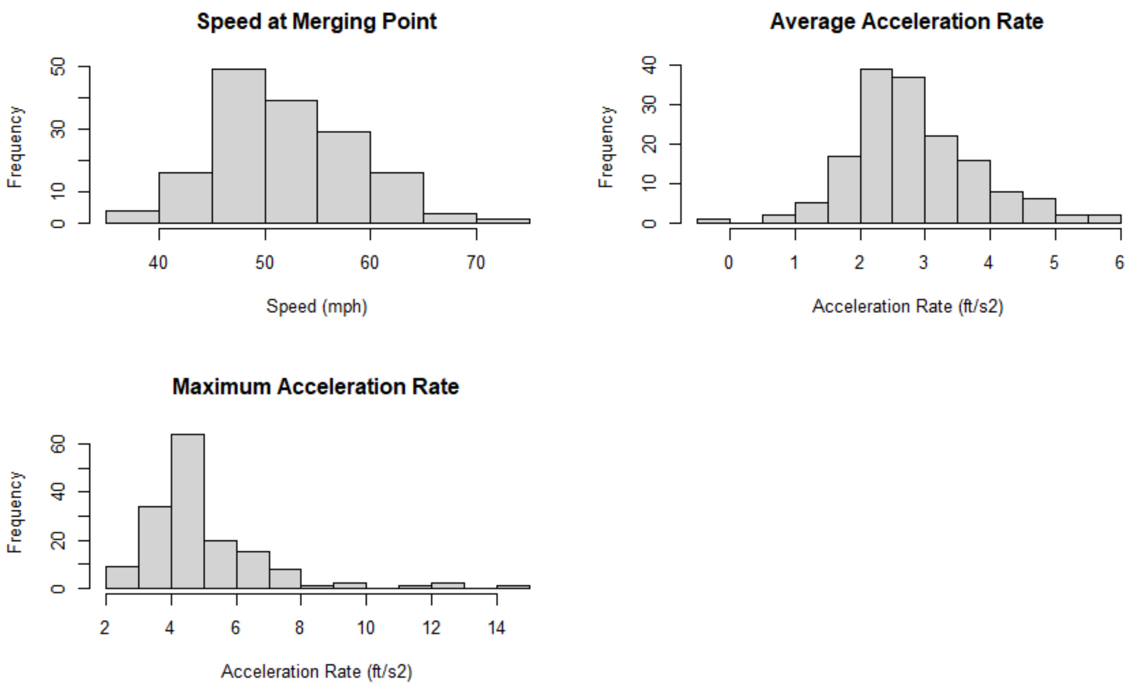


Figure 4-45.Distribution of Parameters for Site CA-1

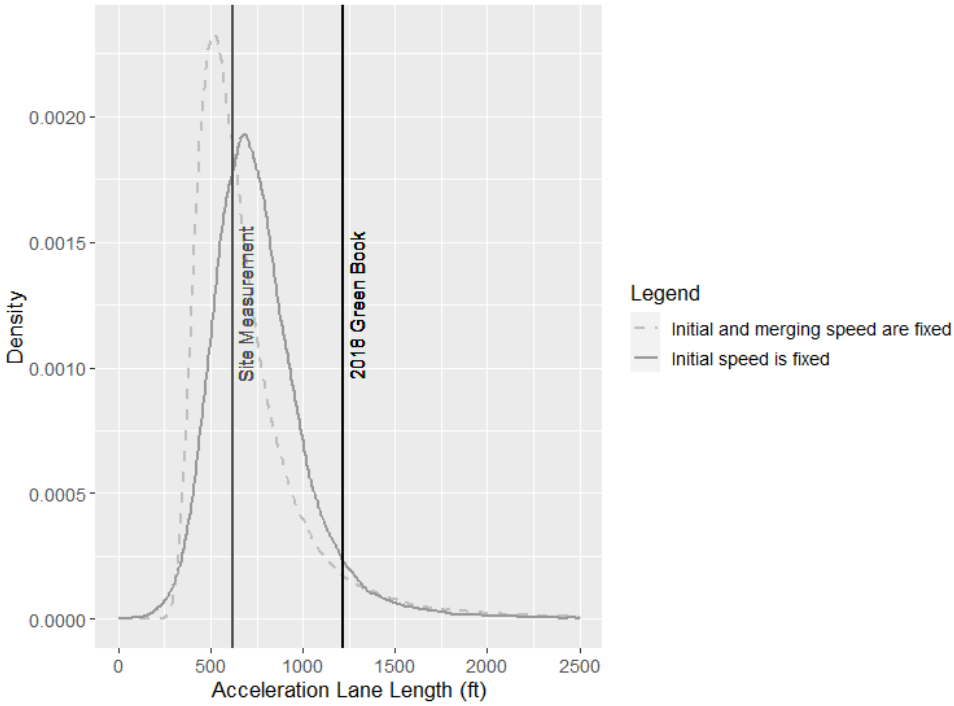


Figure 4-46. Frequency Distribution of Acceleration Lane Length based on Average Acceleration Rate for Site CA-1.

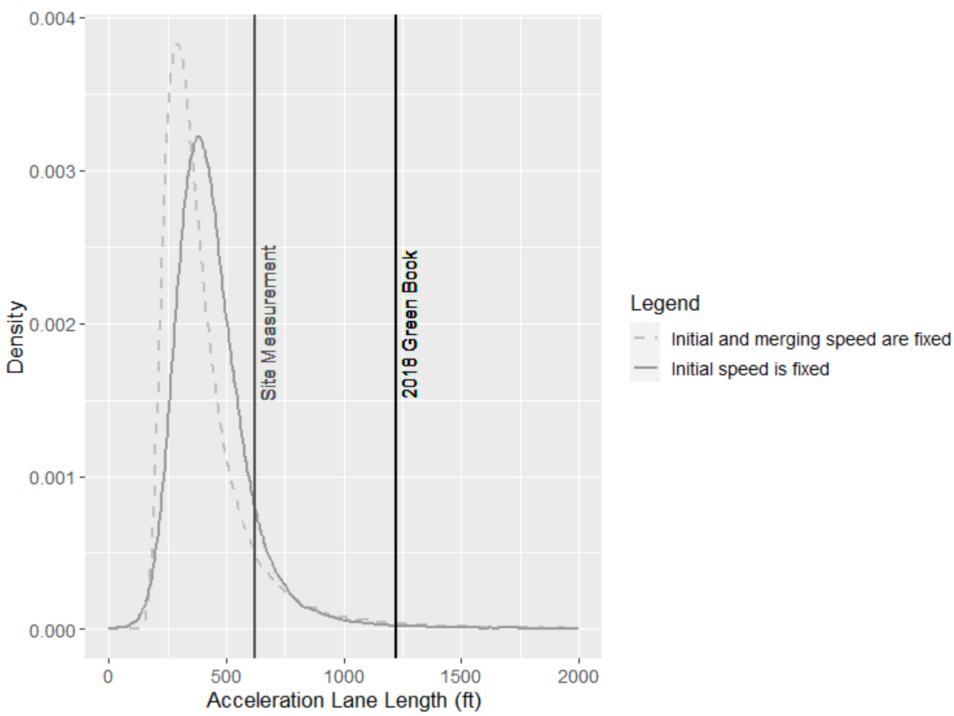


Figure 4-47. Frequency Distribution of Acceleration Lane Length based on Maximum Acceleration Rate for Site CA-1.

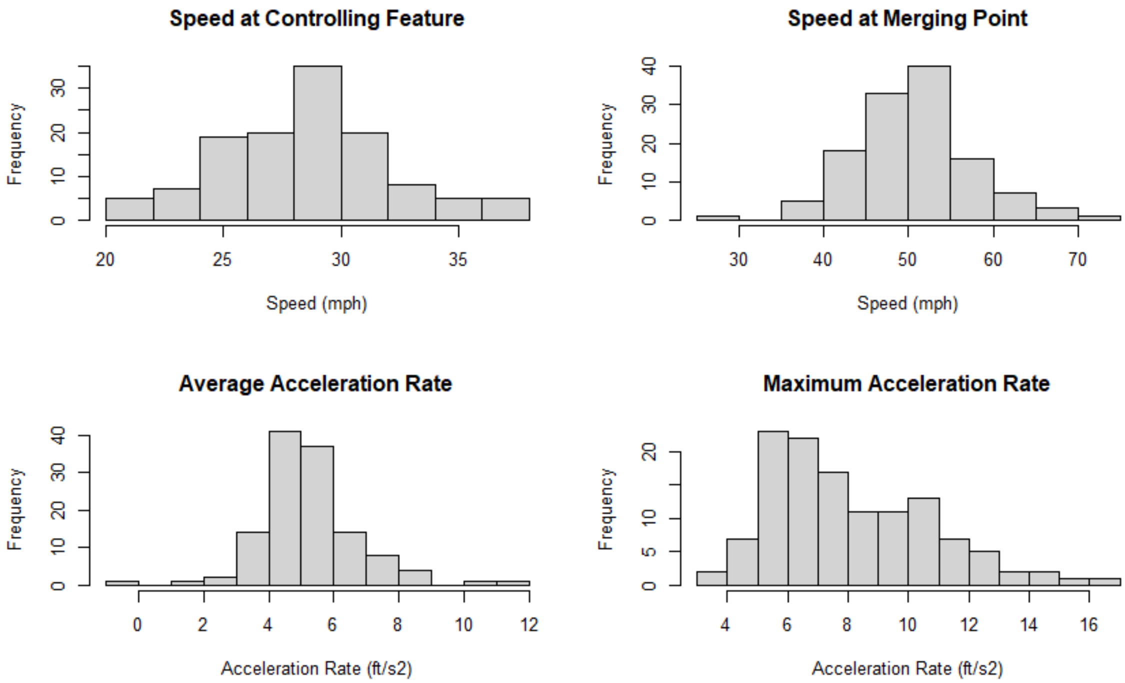


Figure 4-48. Distribution of Parameters for Site CA-2.

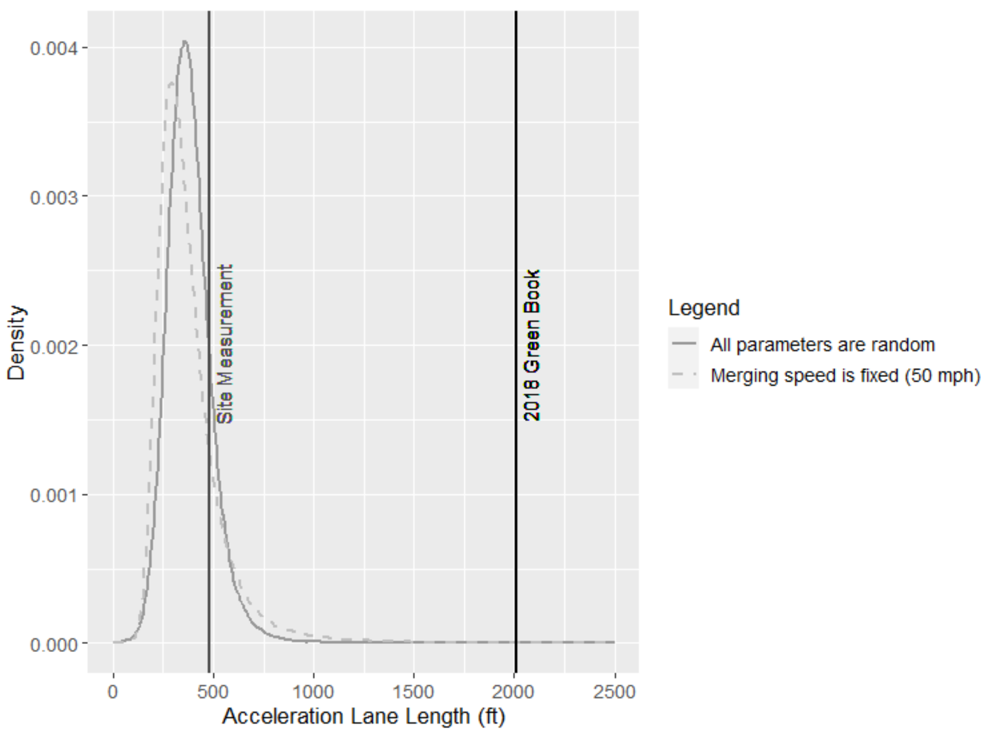


Figure 4-49. Frequency Distribution of Acceleration Lane Length based on Average Acceleration Rate for Site CA-2.

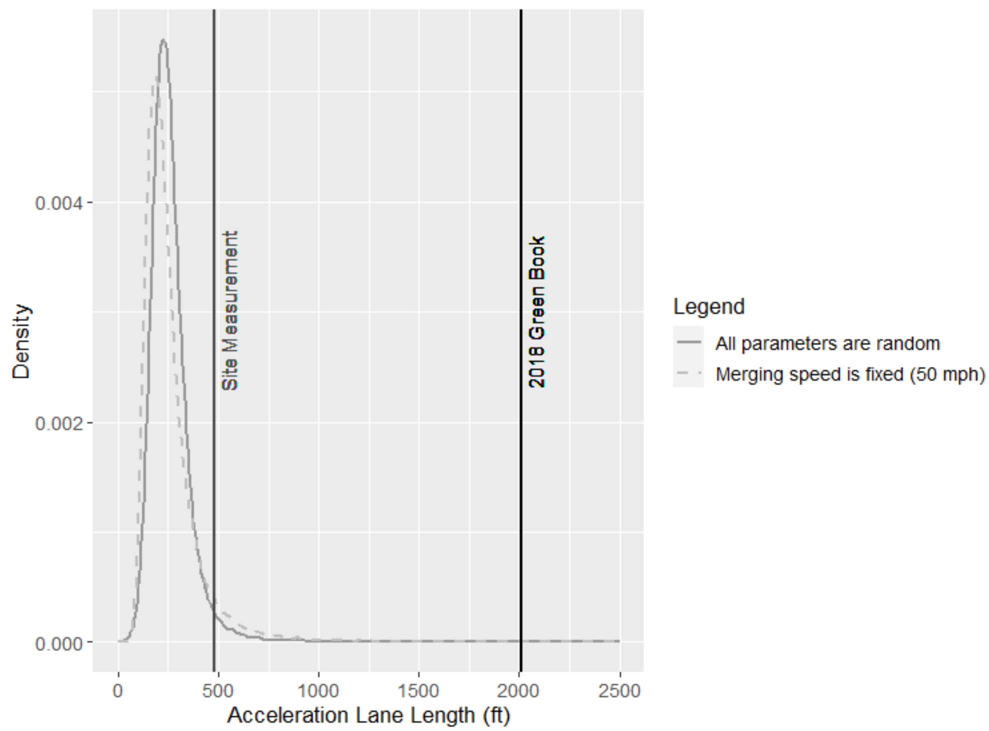


Figure 4-50. Frequency Distribution of Acceleration Lane Length based on Maximum Acceleration Rate for Site CA-2.

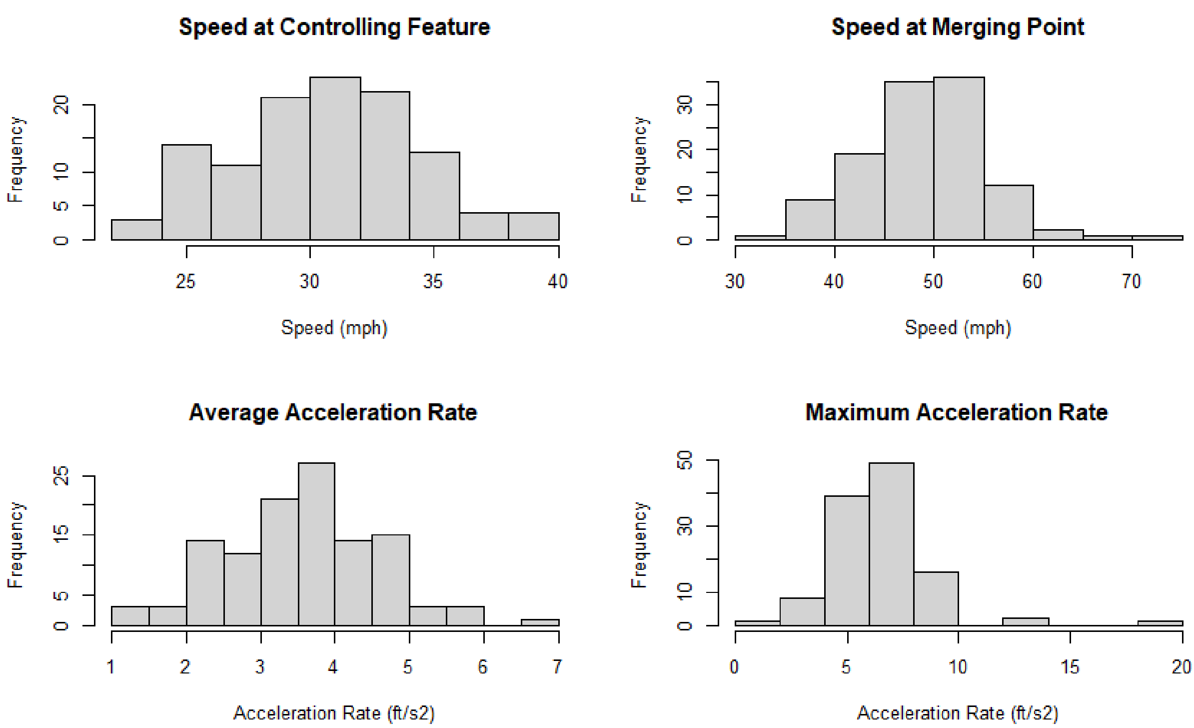


Figure 4-51. Distribution of Parameters for Site CA-3.

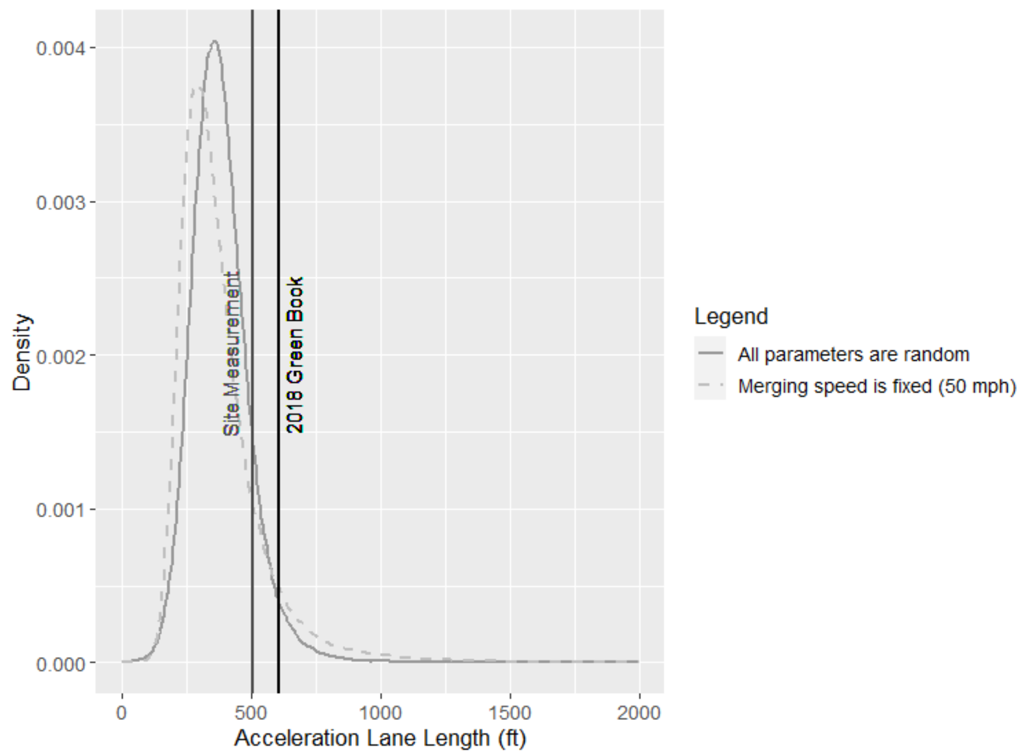


Figure 4-52. Frequency Distribution of Acceleration Lane Length based on Average Acceleration Rate for Site CA-3.

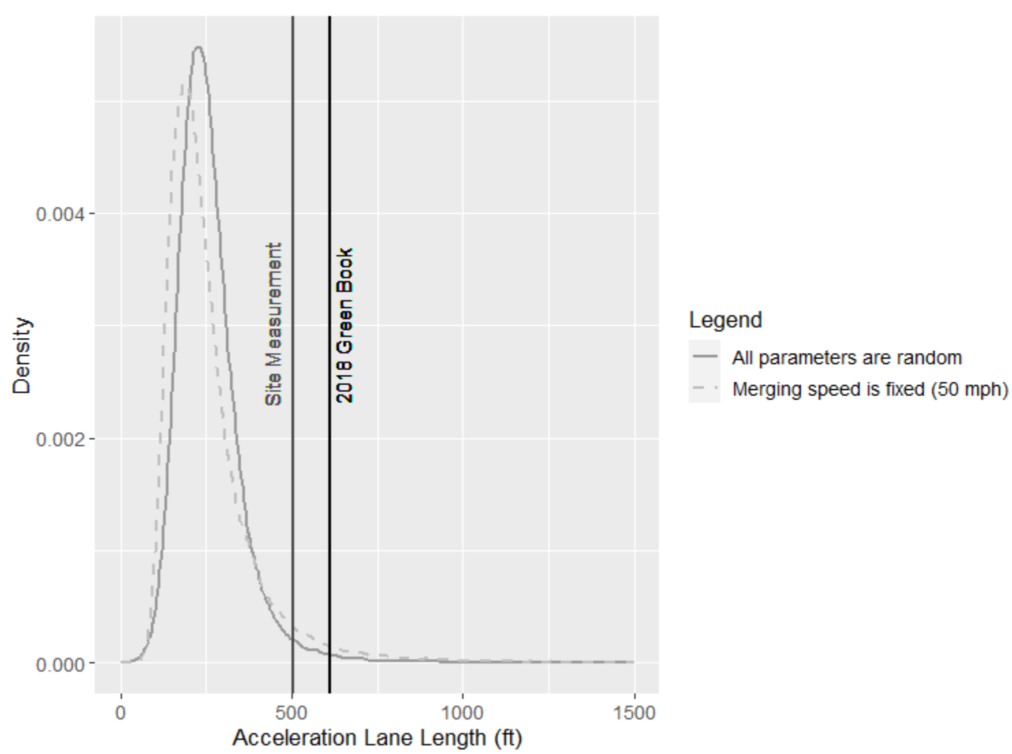


Figure 4-53. Frequency Distribution of Acceleration Lane Length based on Maximum Acceleration Rate for Site CA-3.

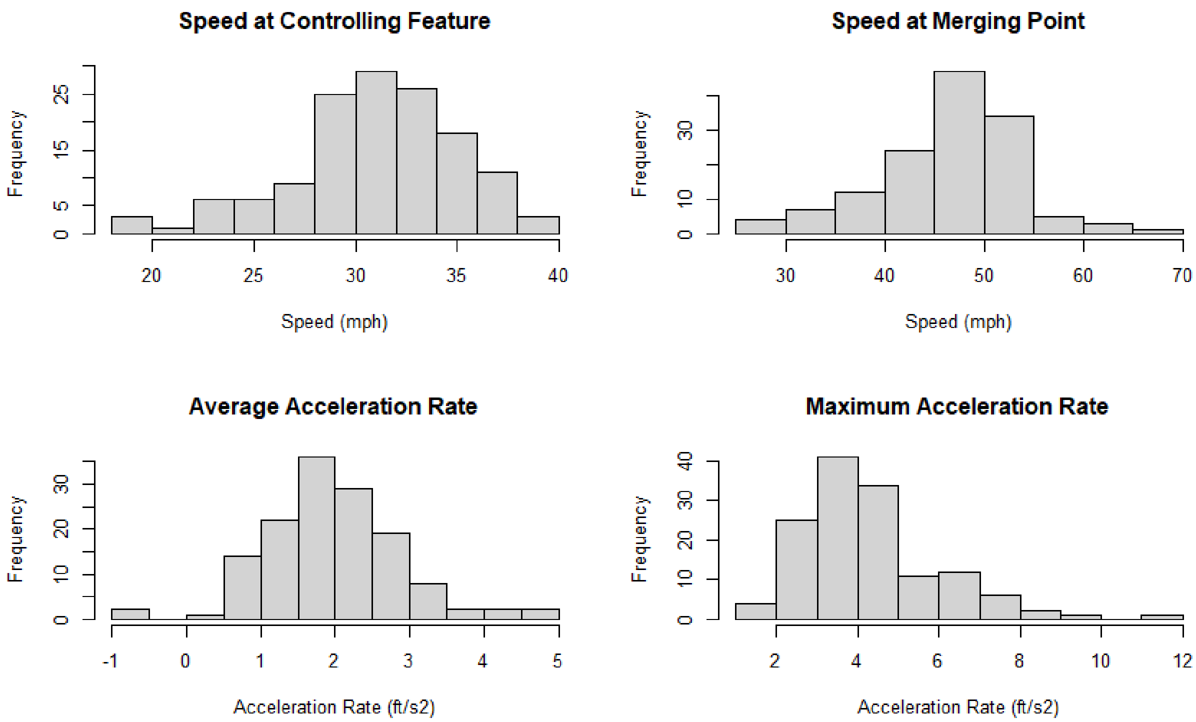


Figure 4-54. Distribution of Parameters for Site CA-4.

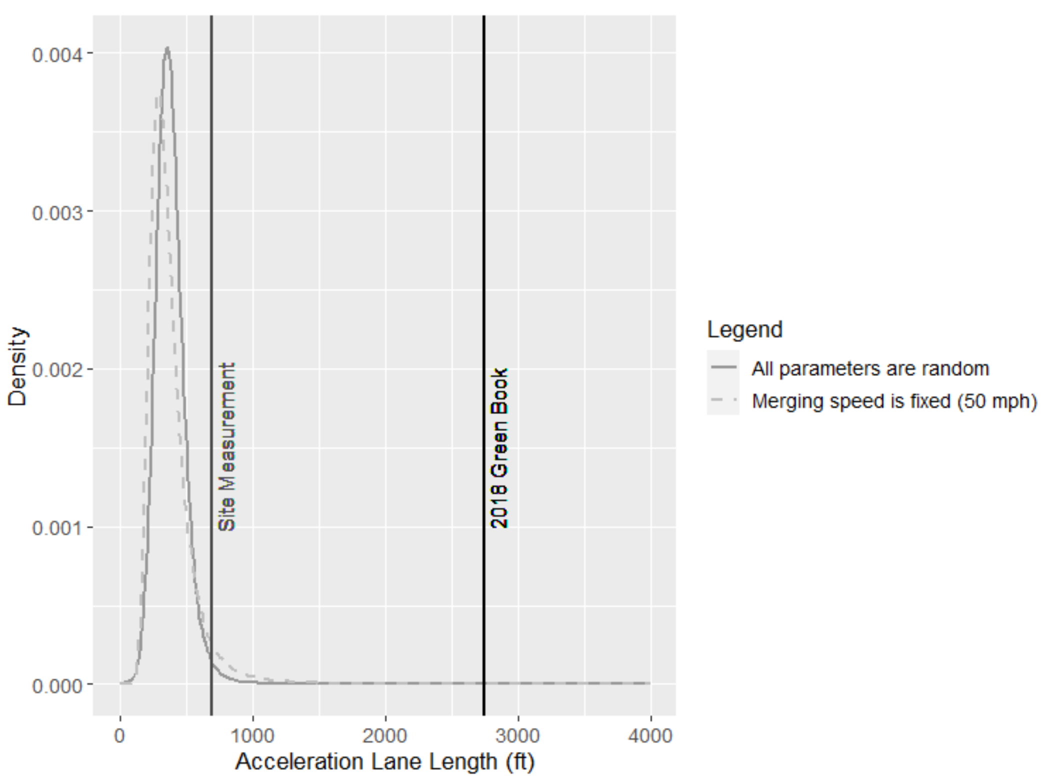


Figure 4-55. Frequency Distribution of Acceleration Lane Length based on Average Acceleration Rate for Site CA-4.

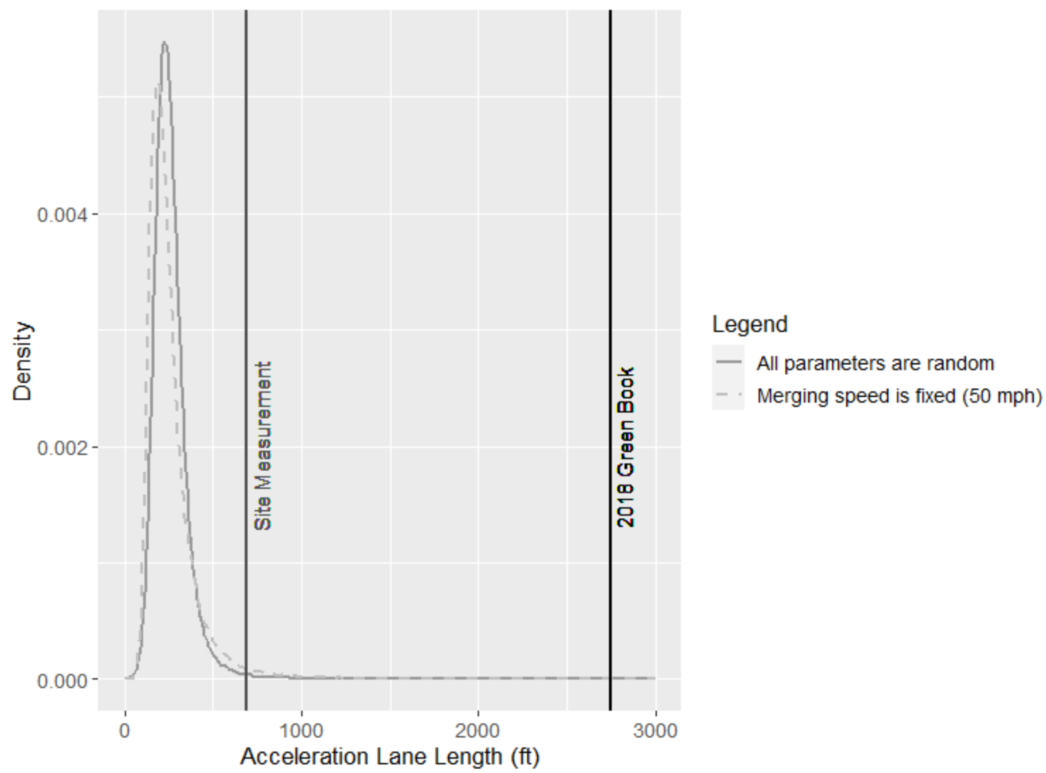


Figure 4-56. Frequency Distribution of Acceleration Lane Length based on Maximum Acceleration Rate for Site CA-4.

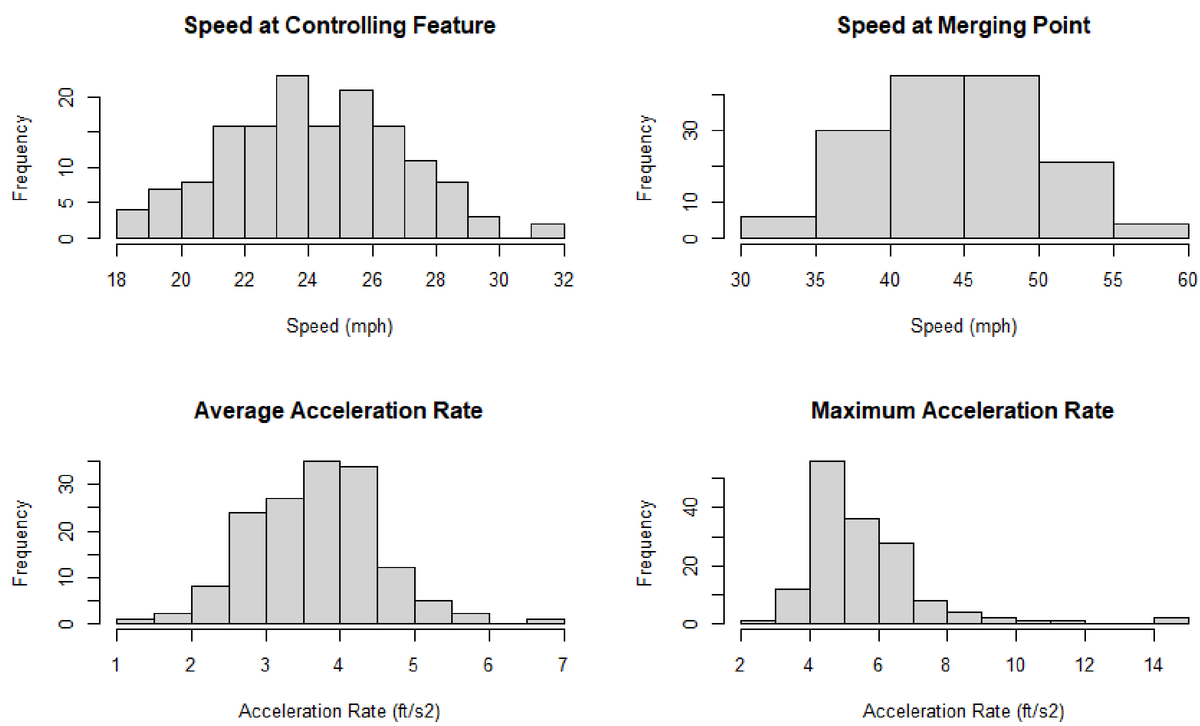


Figure 4-57. Distribution of Parameters for Site CA-5.

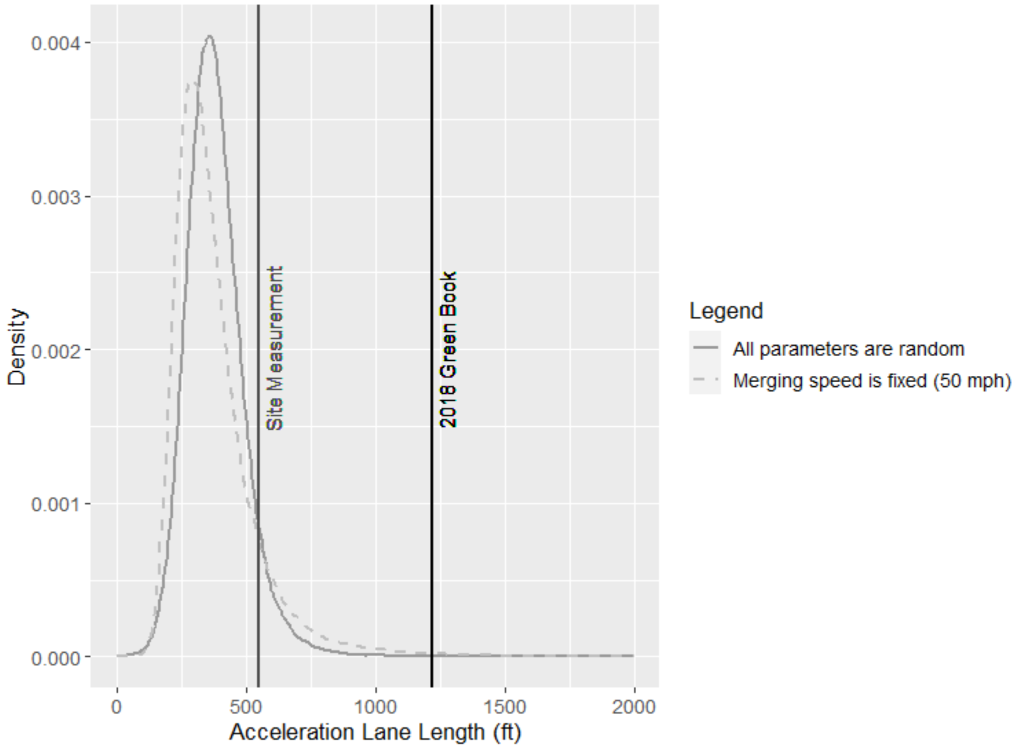


Figure 4-58. Frequency Distribution of Acceleration Lane Length based on Average Acceleration Rate for Site CA-5.

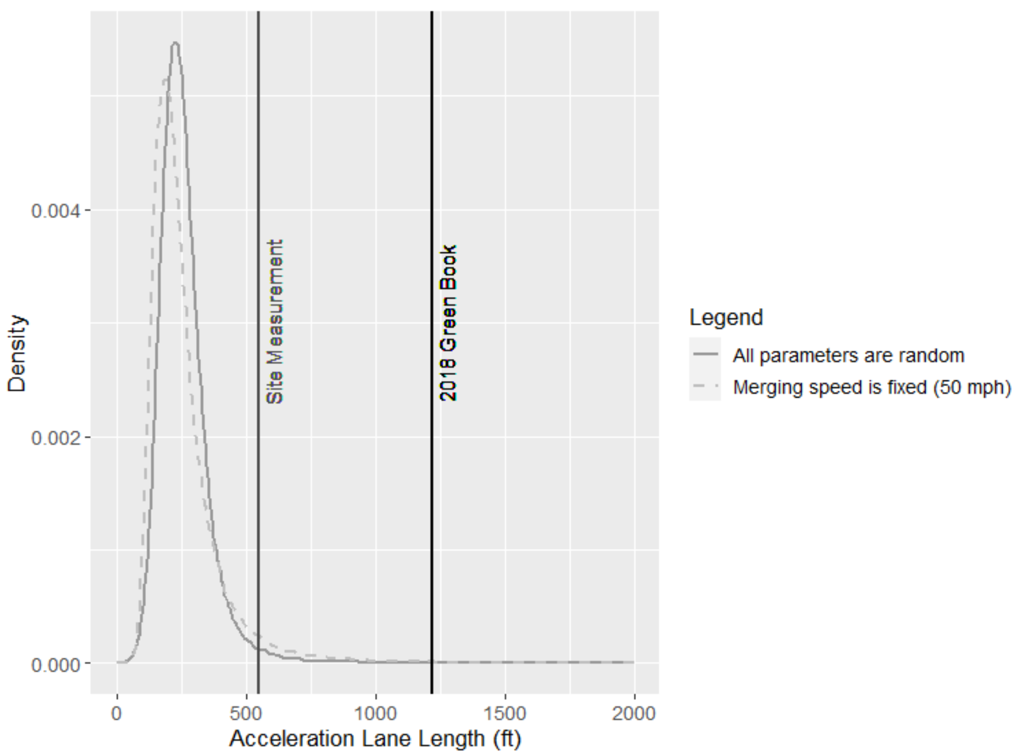


Figure 4-59. Frequency Distribution of Acceleration Lane Length based on Maximum Acceleration Rate for Site CA-5.

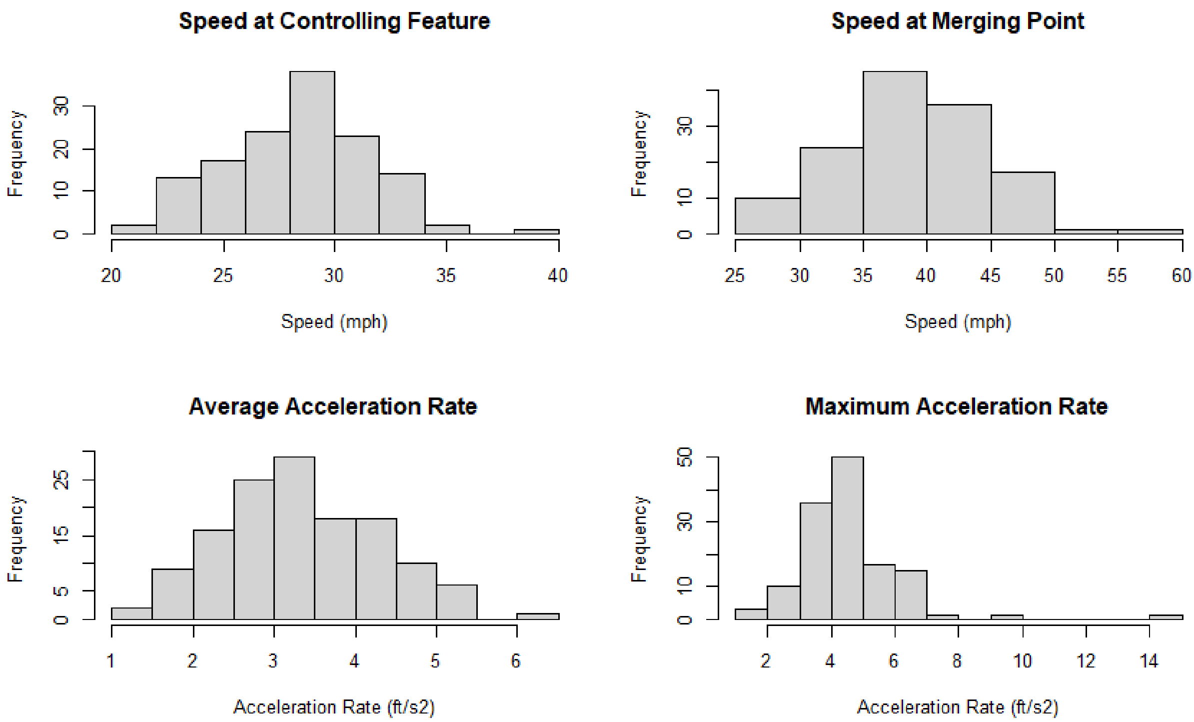


Figure 4-60. Distribution of Parameters for Site CA-6.

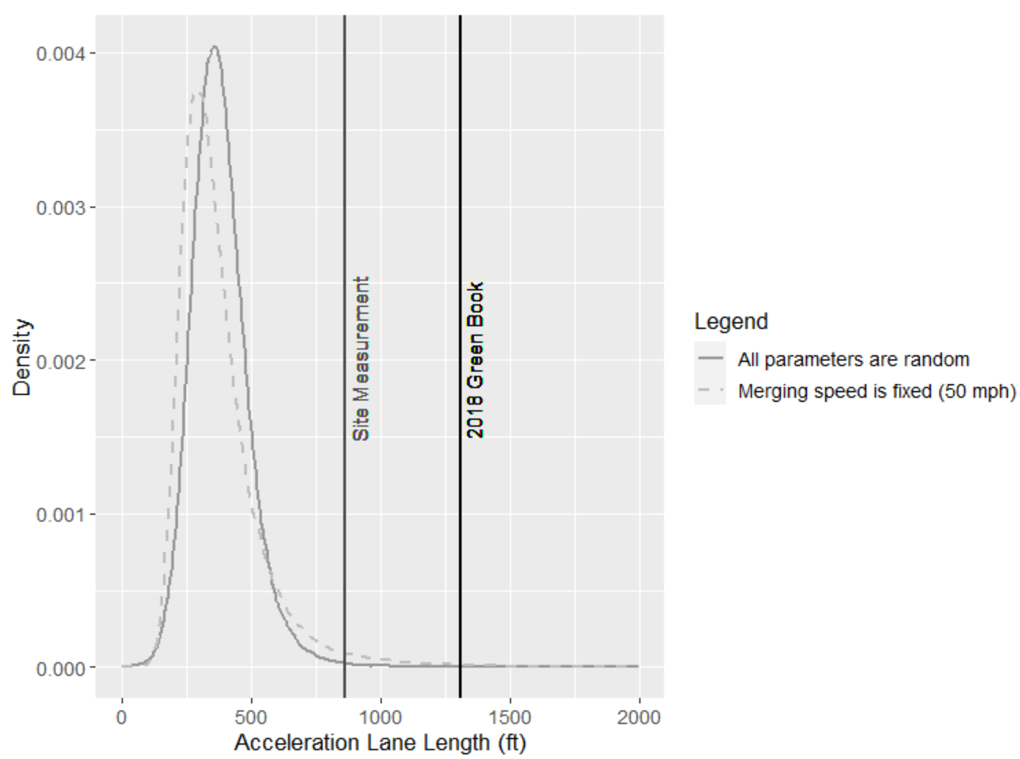


Figure 4-61. Frequency Distribution of Acceleration Lane Length based on Average Acceleration Rate for Site CA-6.

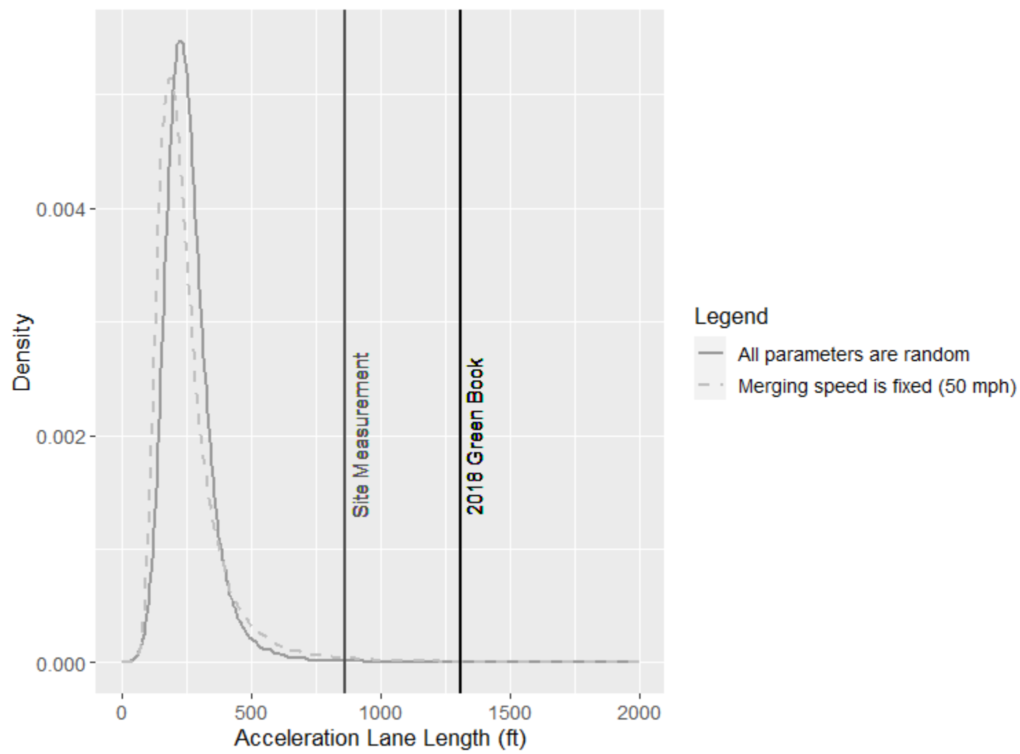


Figure 4-62. Frequency Distribution of Acceleration Lane Length based on Maximum Acceleration Rate for Site CA-6.

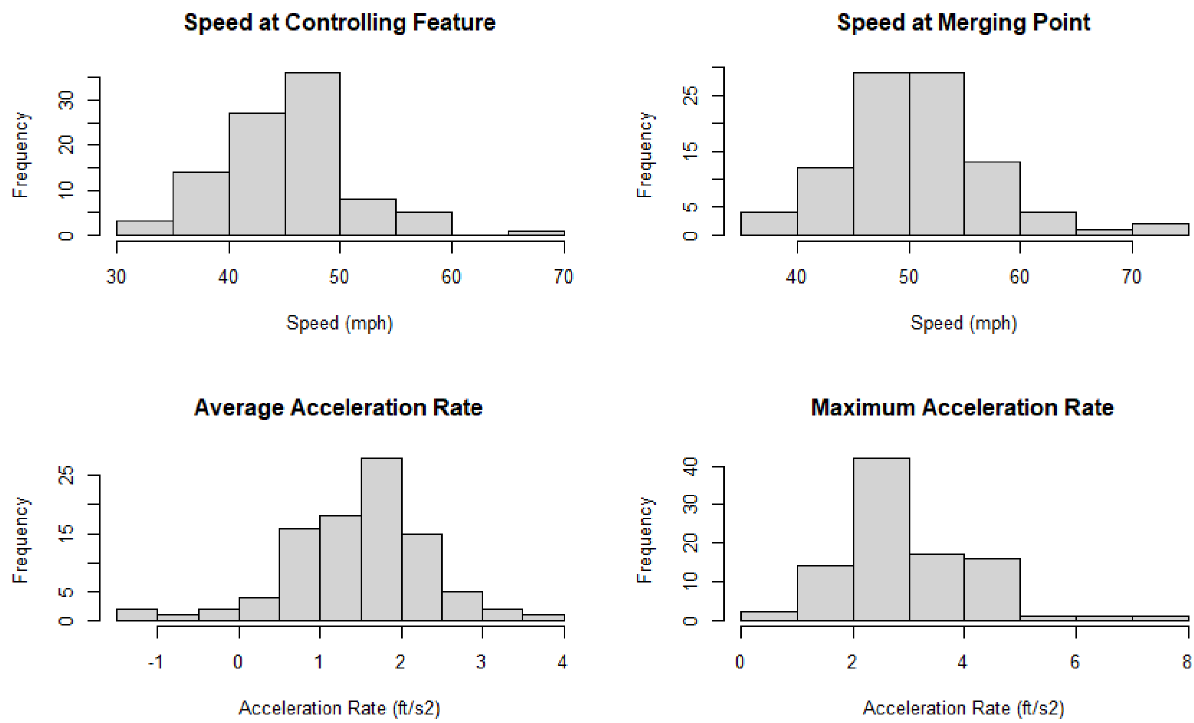


Figure 4-63. Distribution of Parameters for Site CA-7.

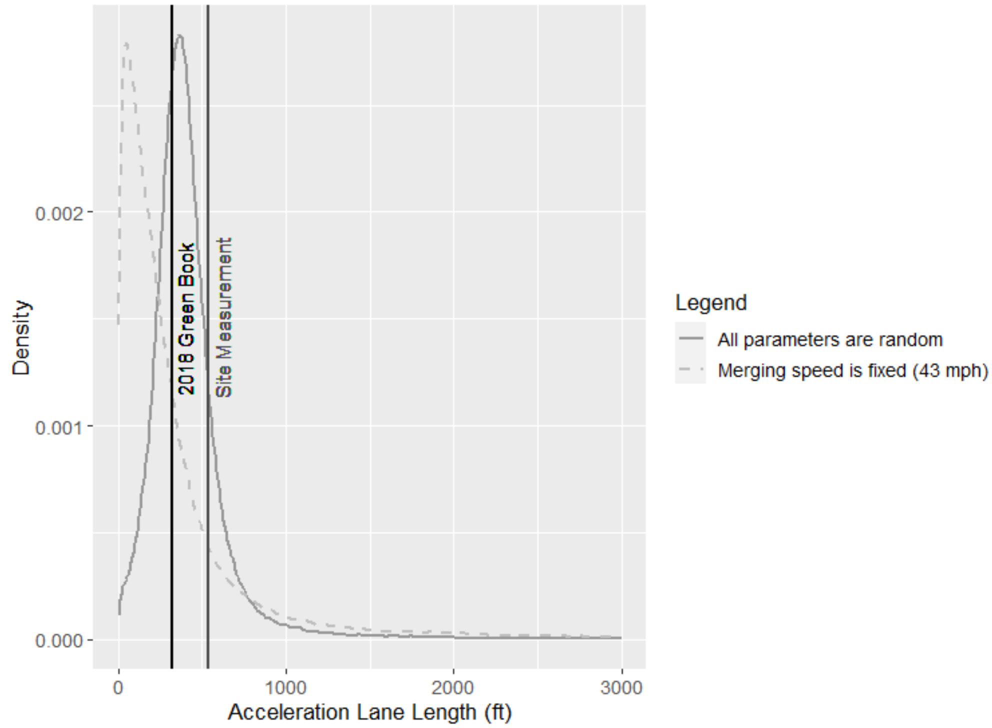


Figure 4-64. Frequency Distribution of Acceleration Lane Length based on Average Acceleration Rate for Site CA-7.

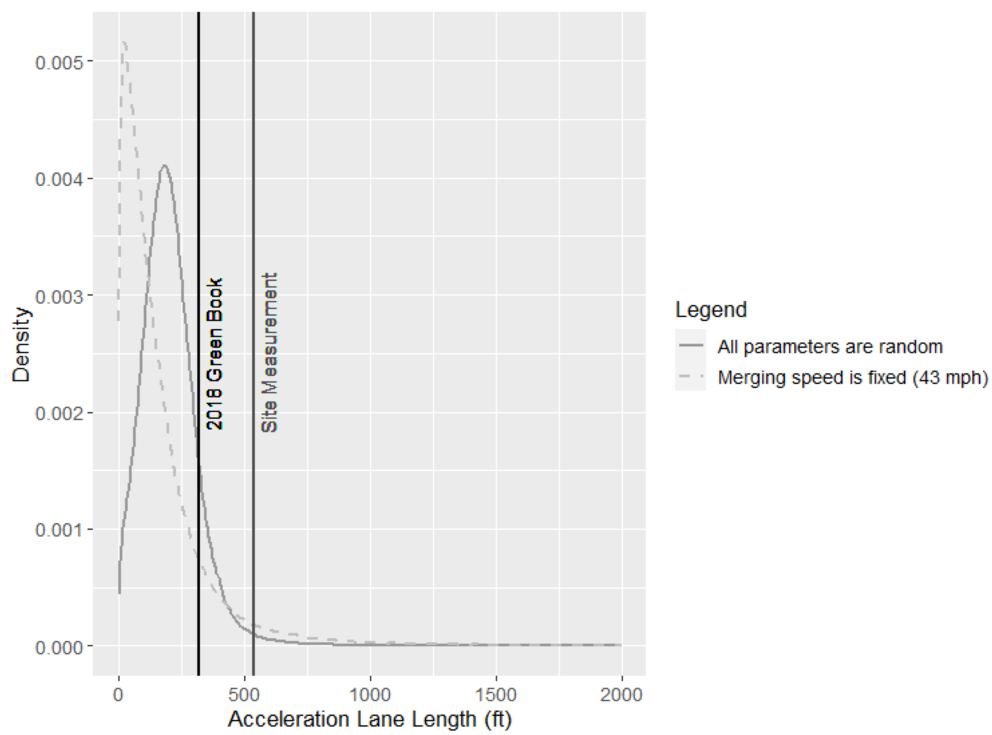


Figure 4-65. Frequency Distribution of Acceleration Lane Length based on Maximum Acceleration Rate for Site CA-7.

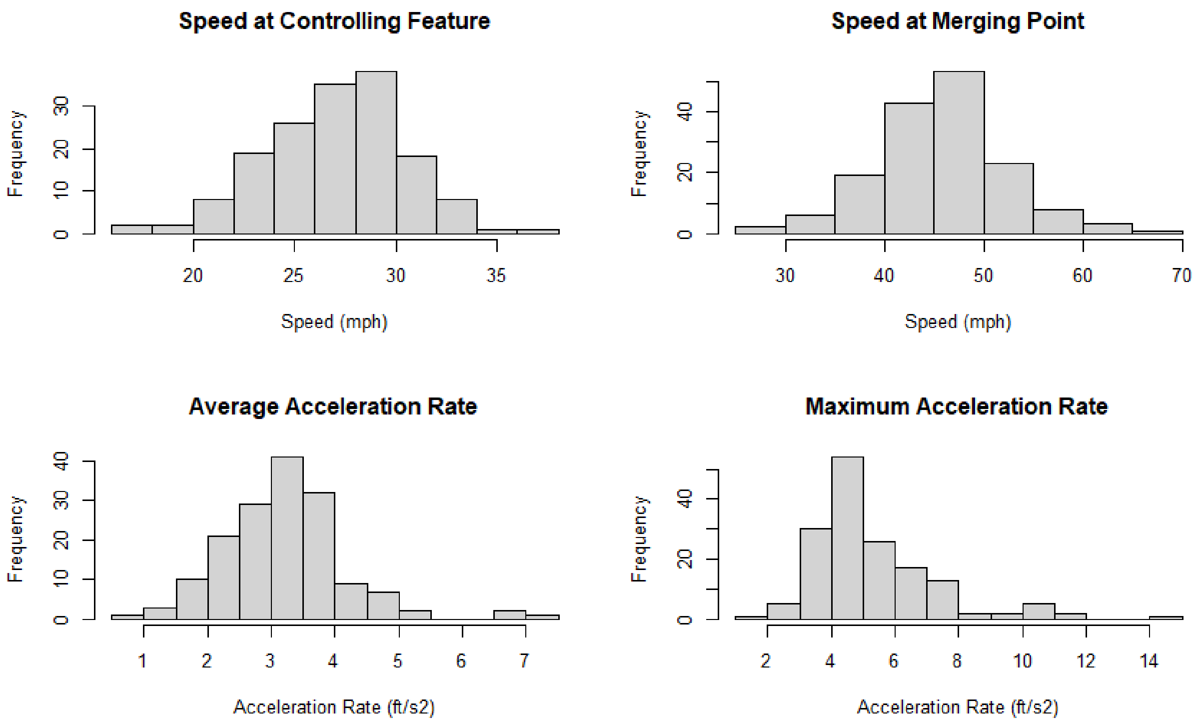


Figure 4-66. Distribution of Parameters for Site CA-9.

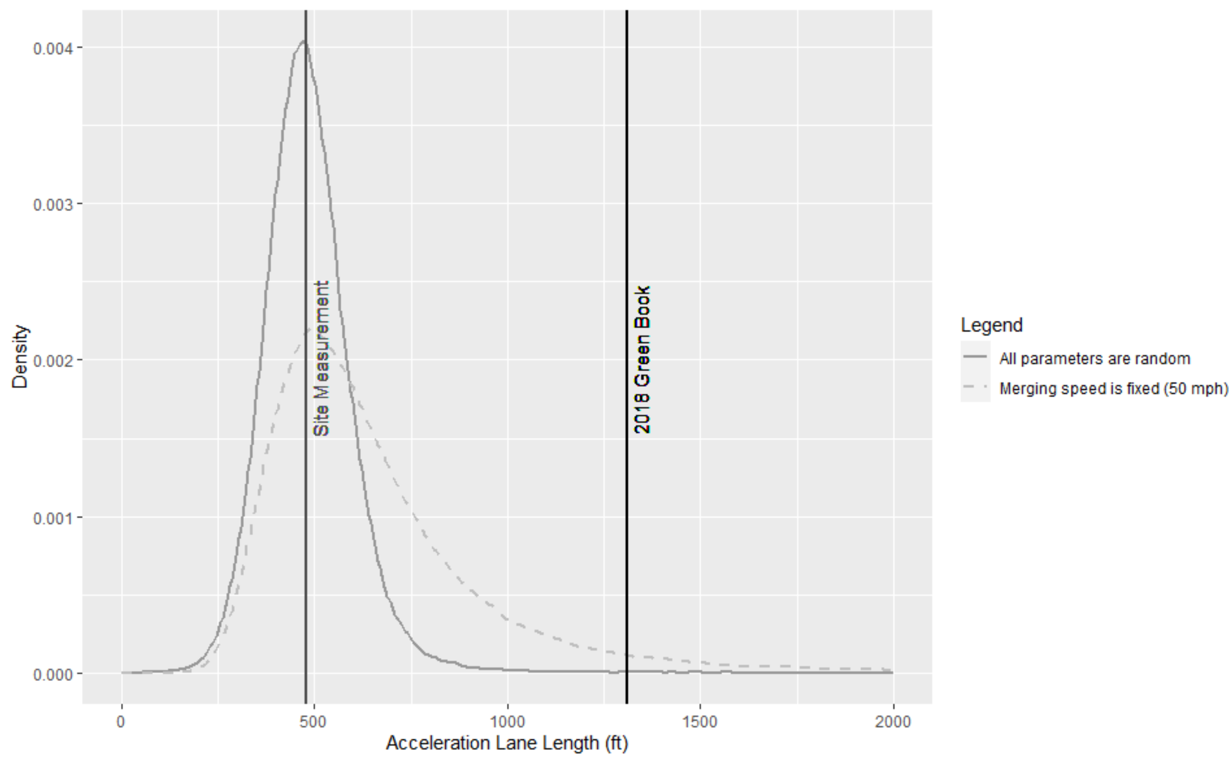


Figure 4-67. Frequency Distribution of Acceleration Lane Length based on Average Acceleration Rate for Site CA-9.

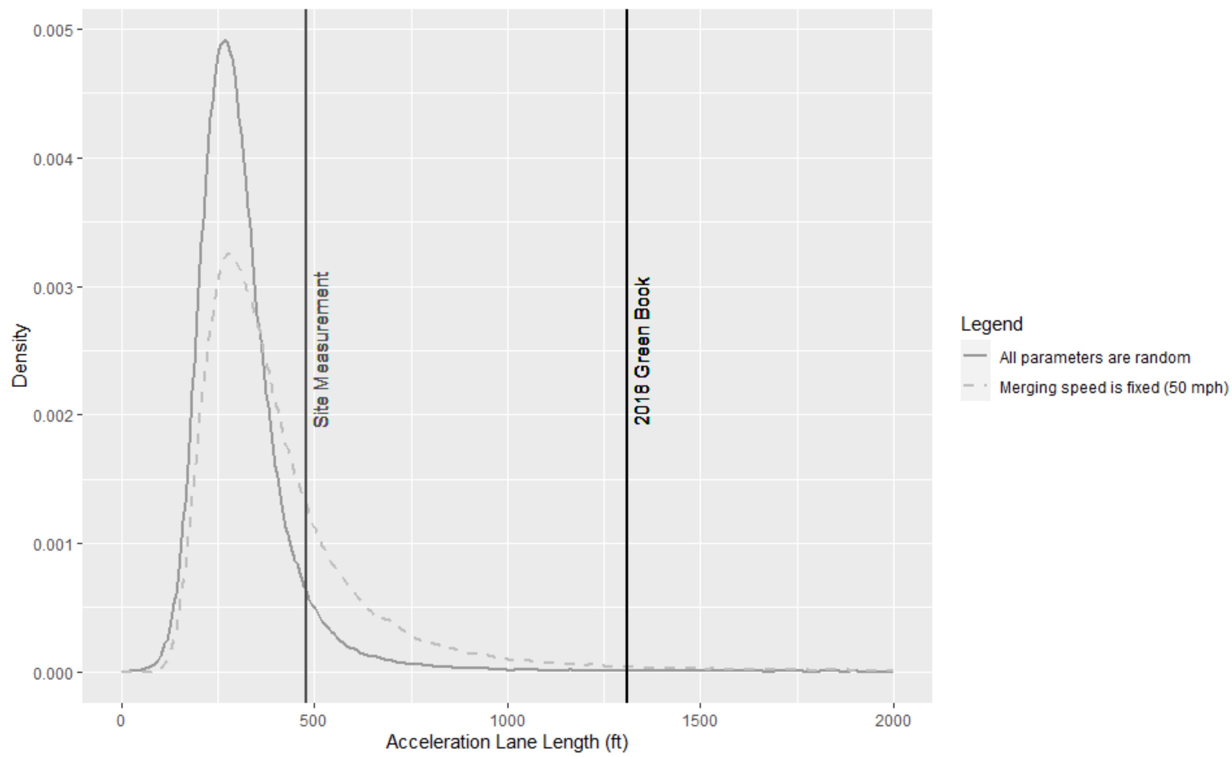


Figure 4-68. Frequency Distribution of Acceleration Lane Length based on Maximum Acceleration Rate for Site CA-9.

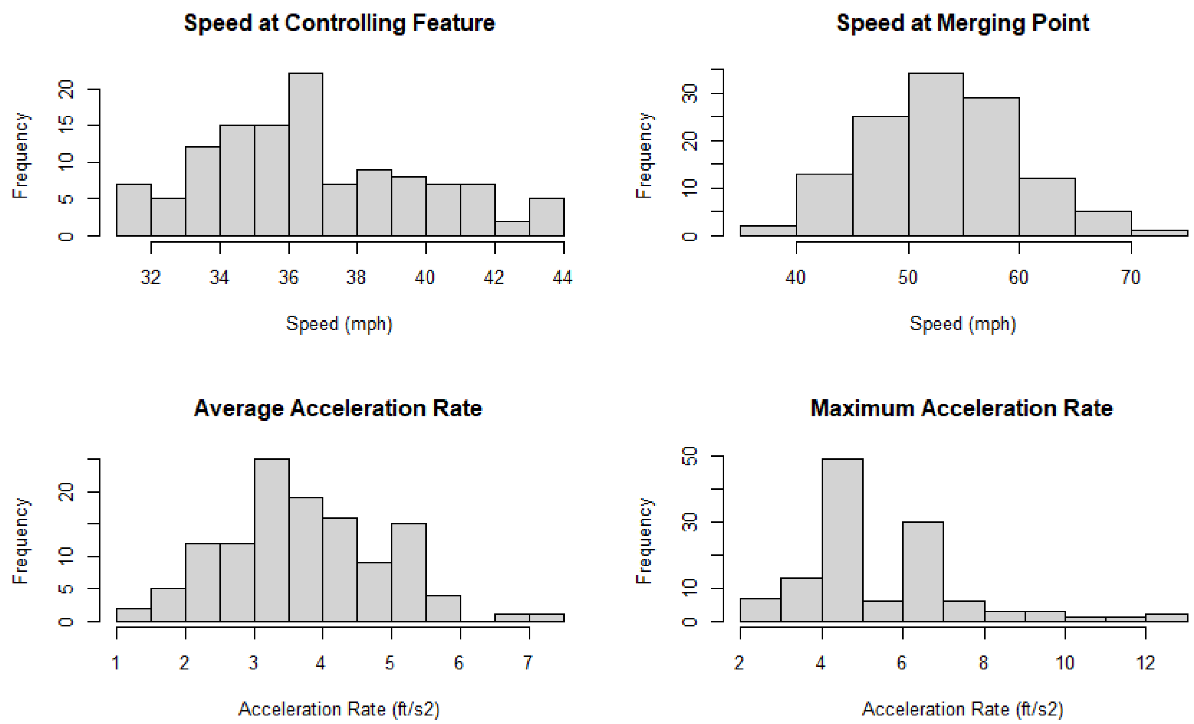


Figure 4-69. Distribution of Parameters for Site MI-1.

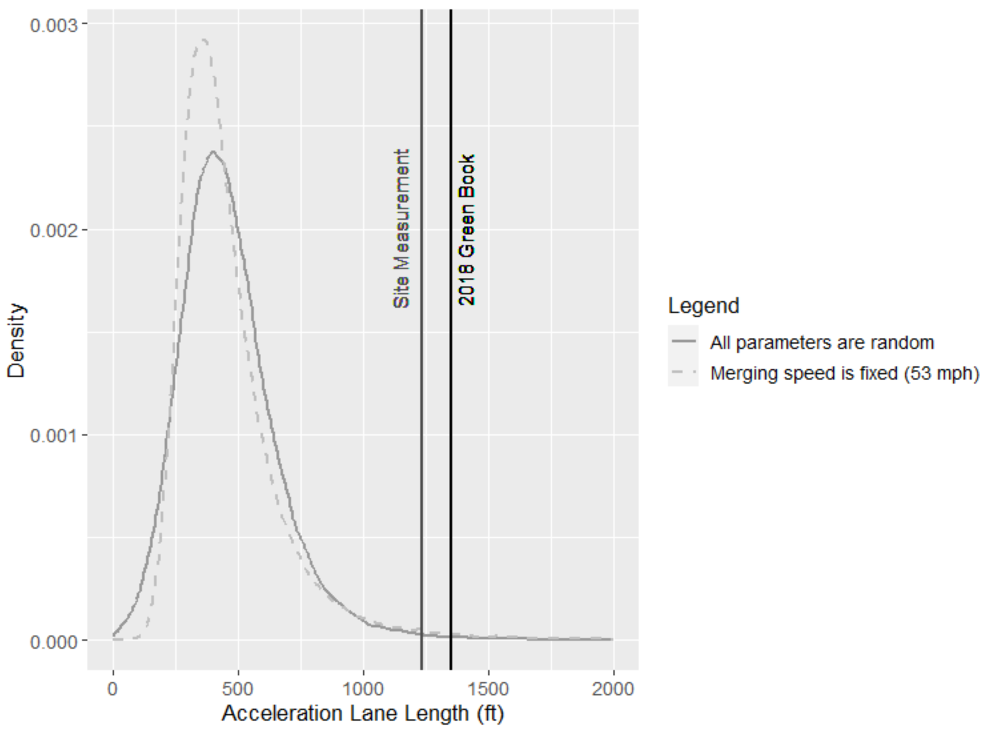


Figure 4-70. Frequency Distribution of Acceleration Lane Length based on Average Acceleration Rate for Site MI-1.

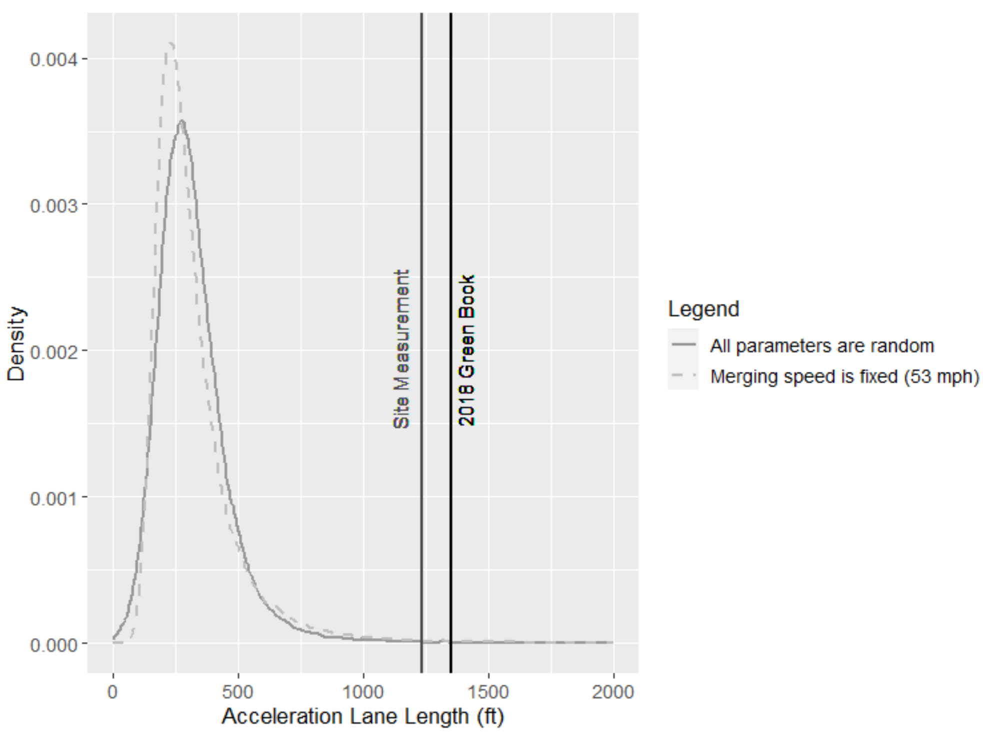


Figure 4-71. Frequency Distribution of Acceleration Lane Length based on Maximum Acceleration Rate for Site MI-1.

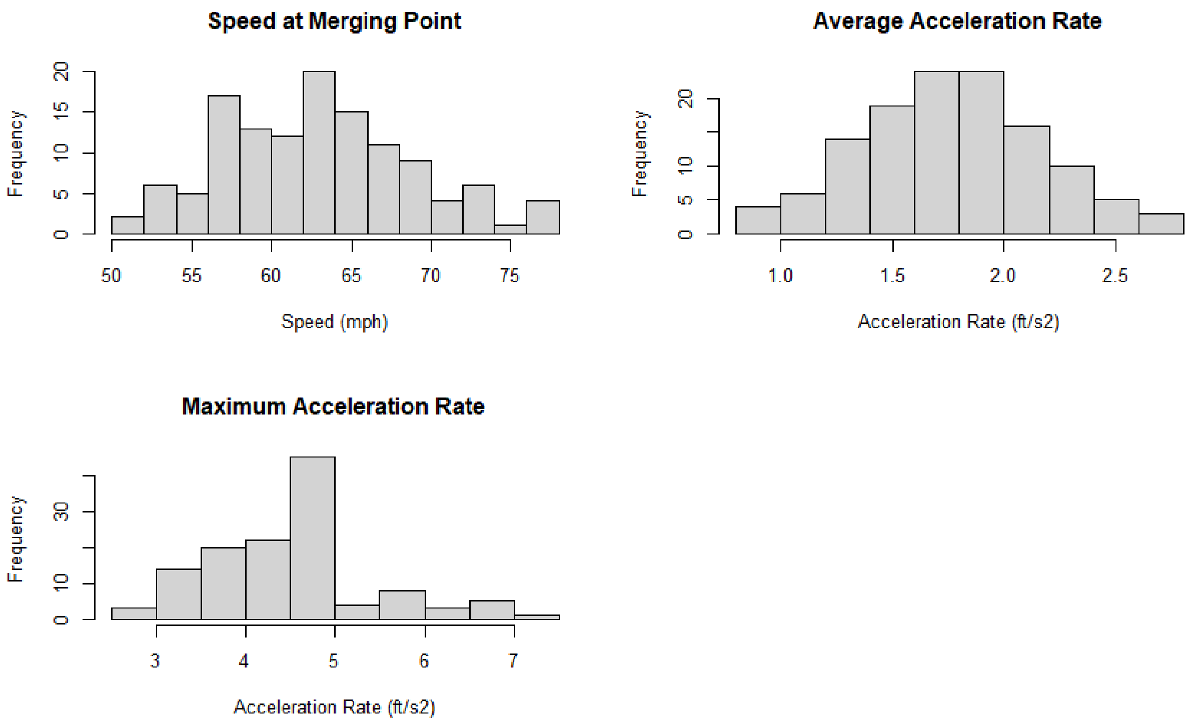


Figure 4-72. Distribution of Parameters for Site MI-2.

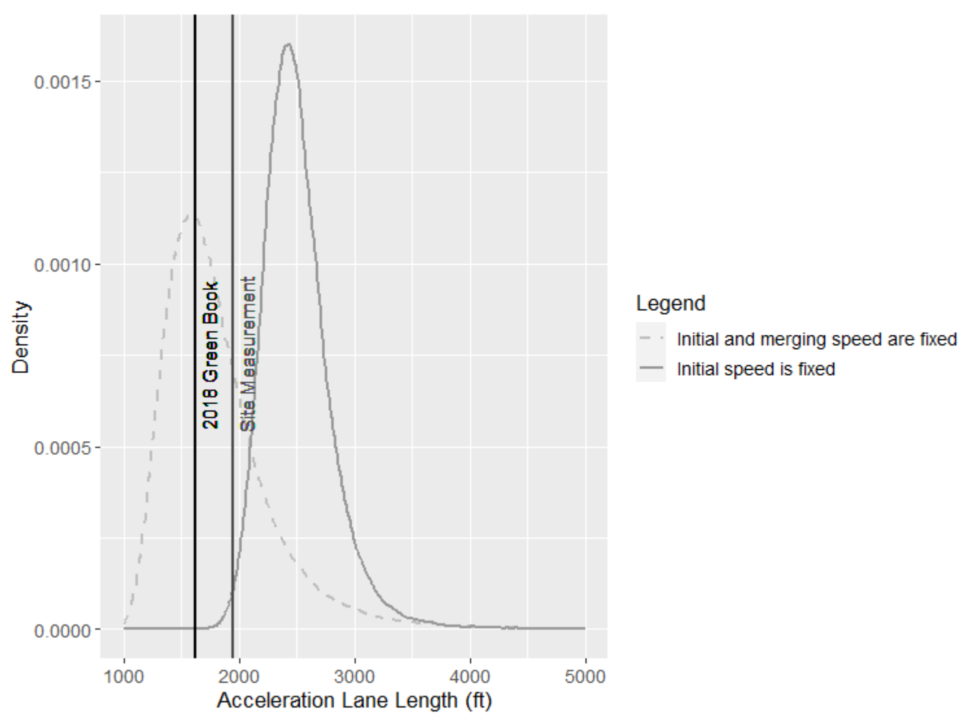


Figure 4-73. Frequency Distribution of Acceleration Lane Length based on Average Acceleration Rate for Site MI-2.

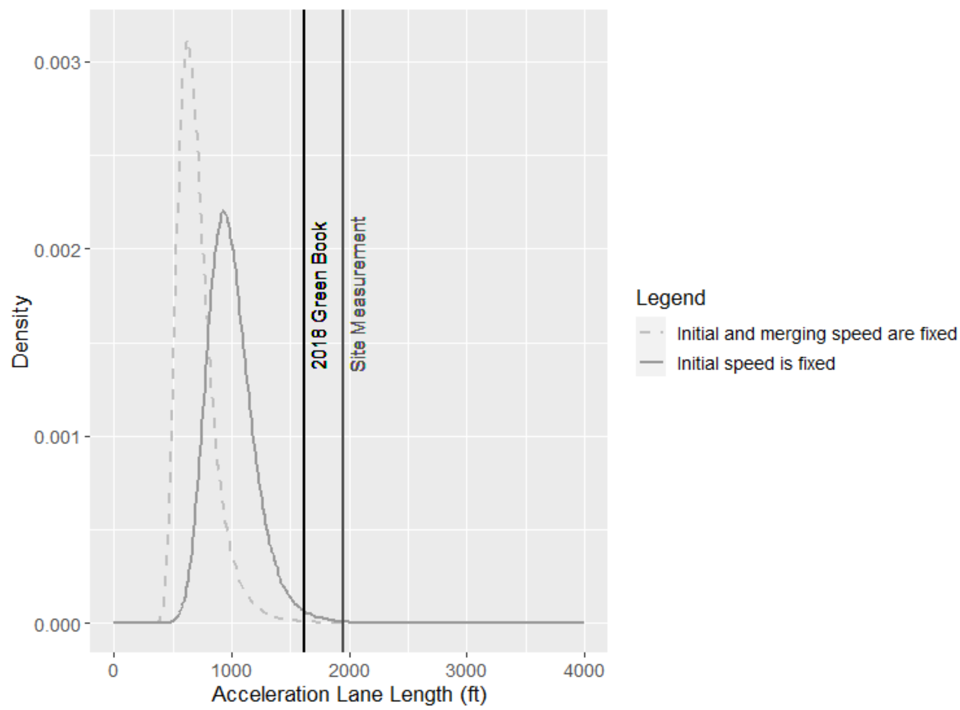


Figure 4-74. Frequency Distribution of Acceleration Lane Length based on Maximum Acceleration Rate for Site MI-2.

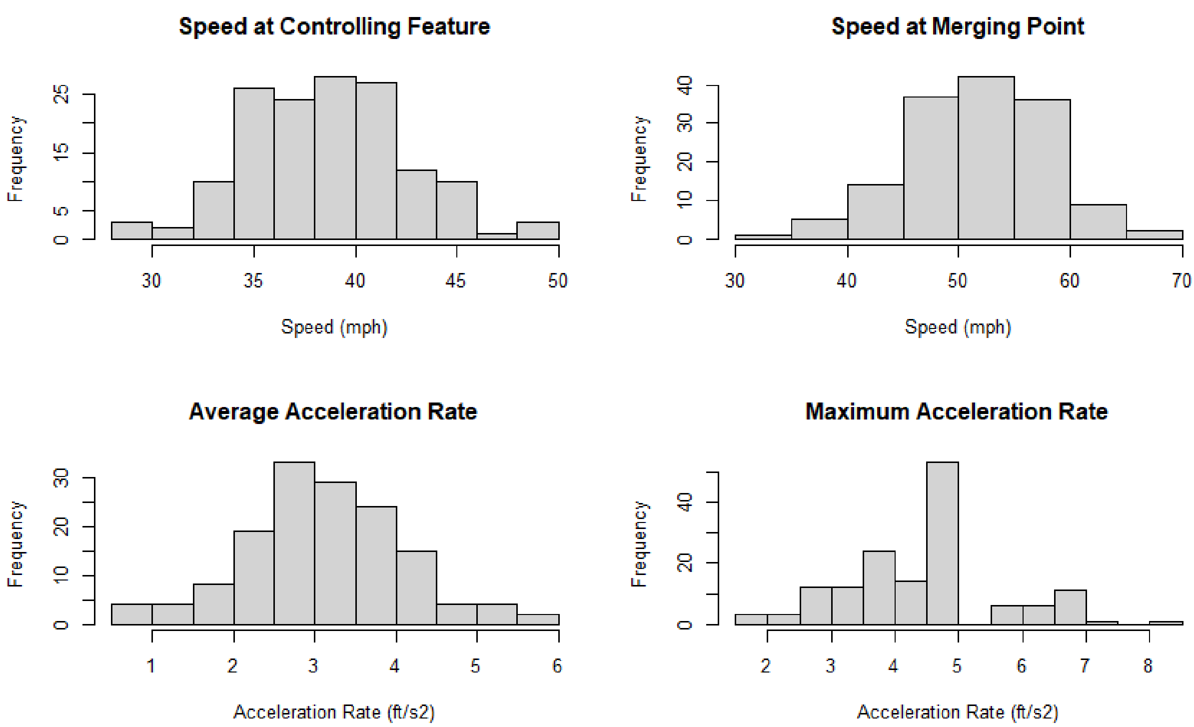


Figure 4-75. Distribution of Parameters for Site MI-3.

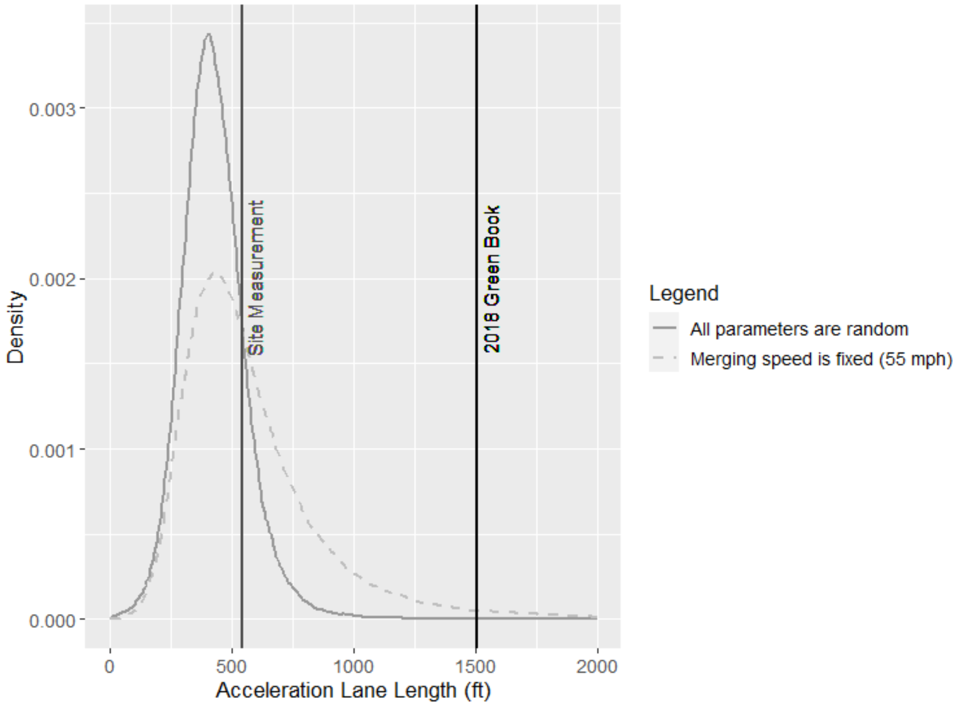


Figure 4-76. Frequency Distribution of Acceleration Lane Length based on Average Acceleration Rate for Site MI-3.

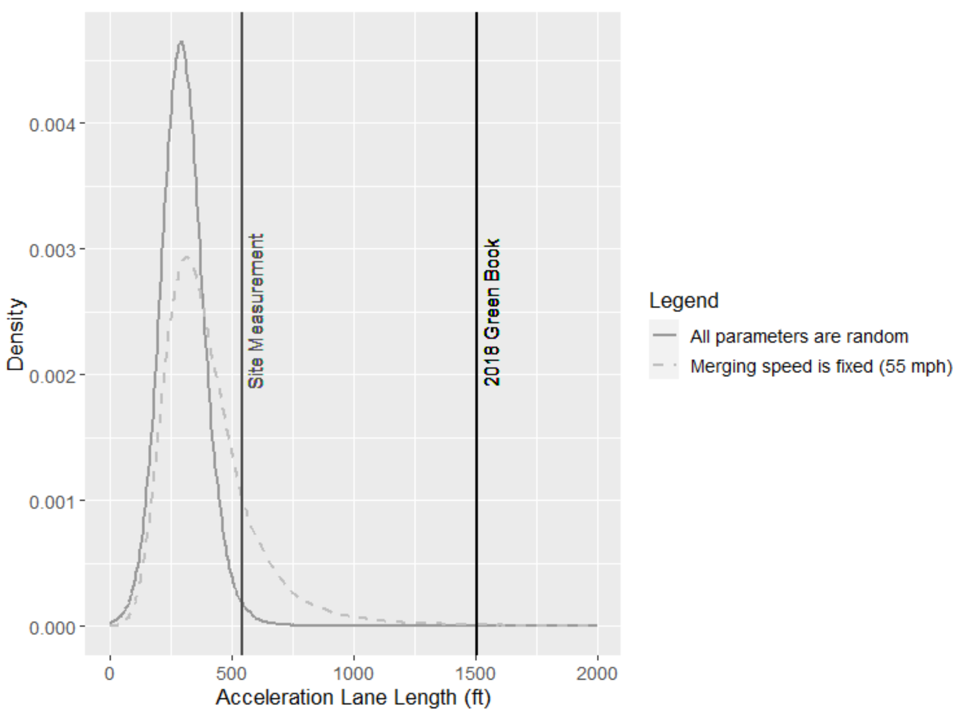


Figure 4-77. Frequency Distribution of Acceleration Lane Length based on Maximum Acceleration Rate for Site MI-3.

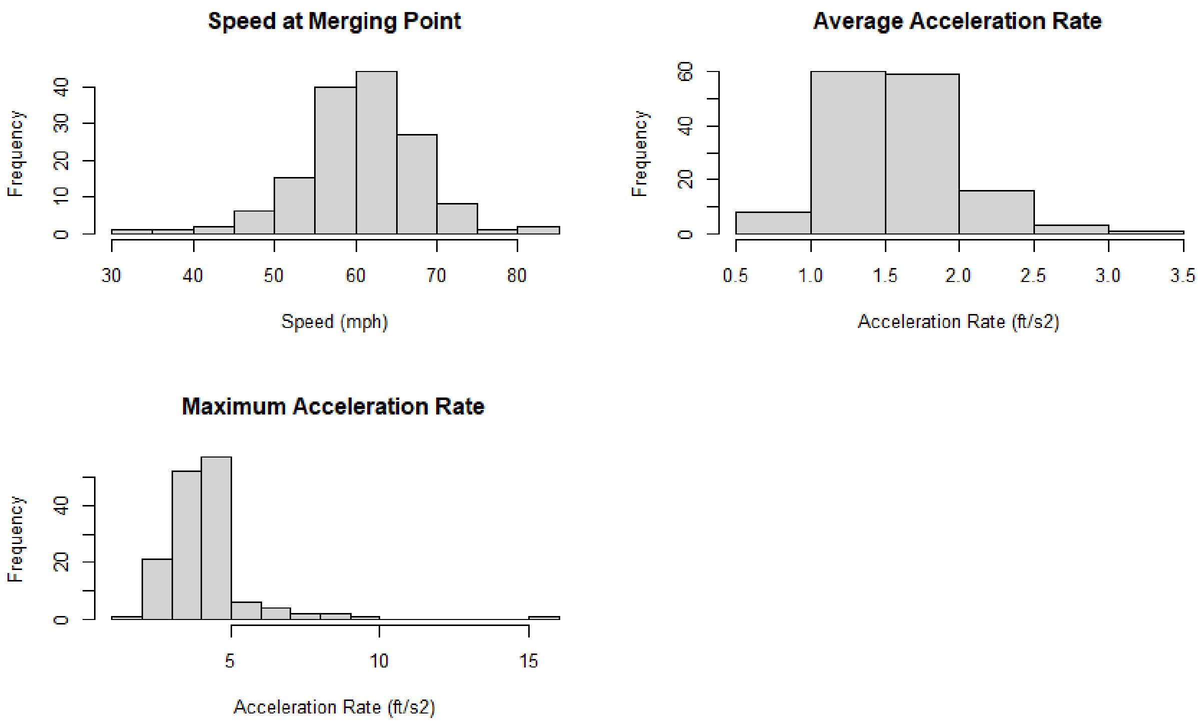


Figure 4-78. Distribution of Parameters for Site MI-4.

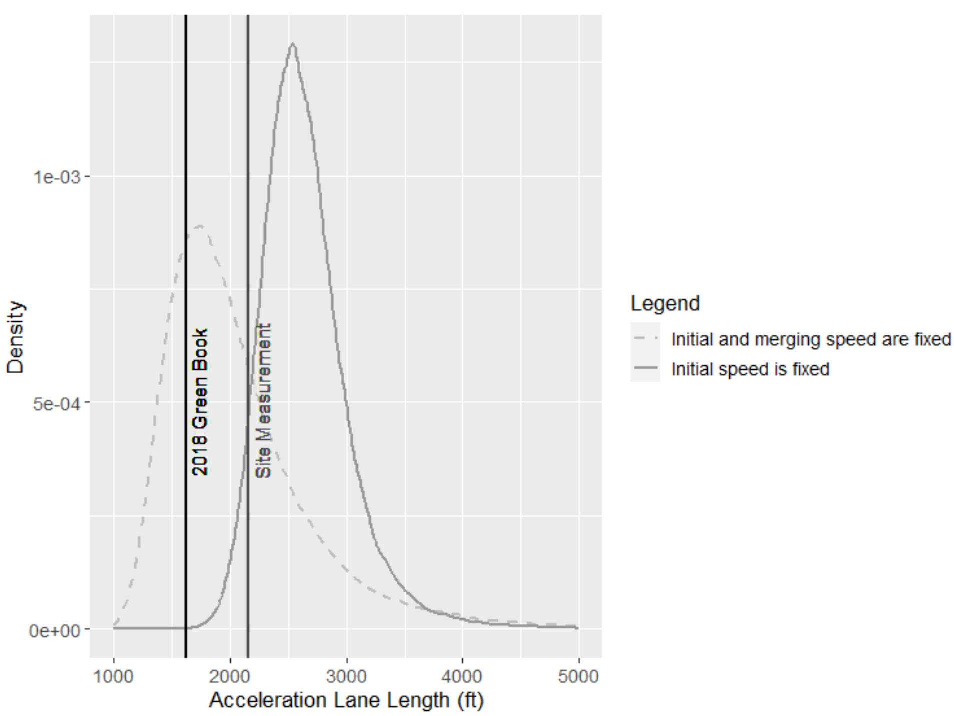


Figure 4-79. Frequency Distribution of Acceleration Lane Length based on Average Acceleration Rate for Site MI-4.

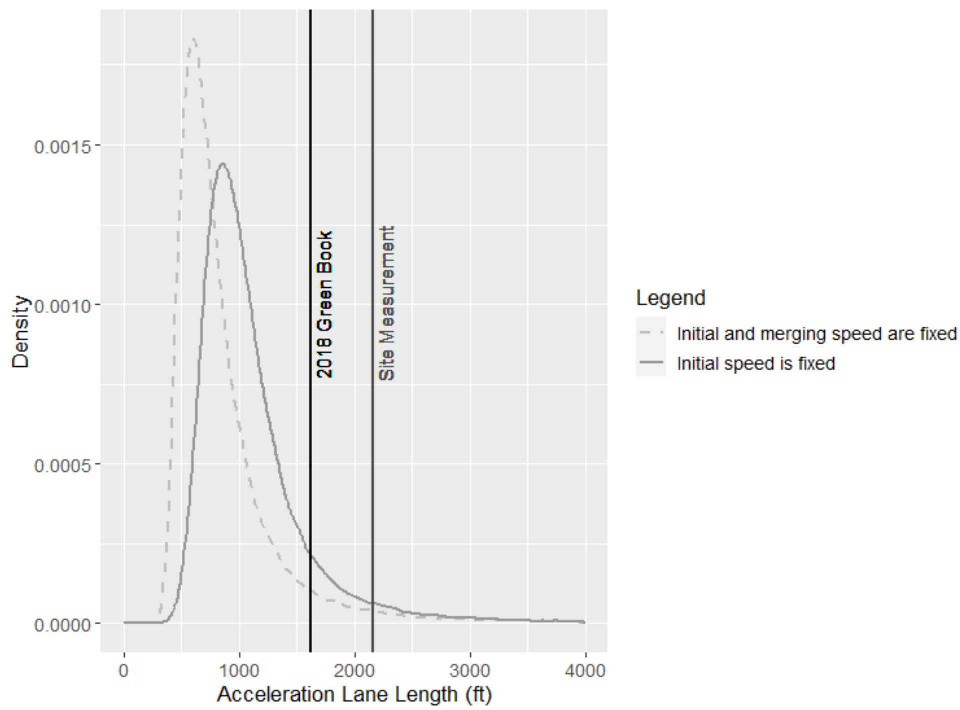


Figure 4-80. Frequency Distribution of Acceleration Lane Length based on Maximum Acceleration Rate for Site MI-4.

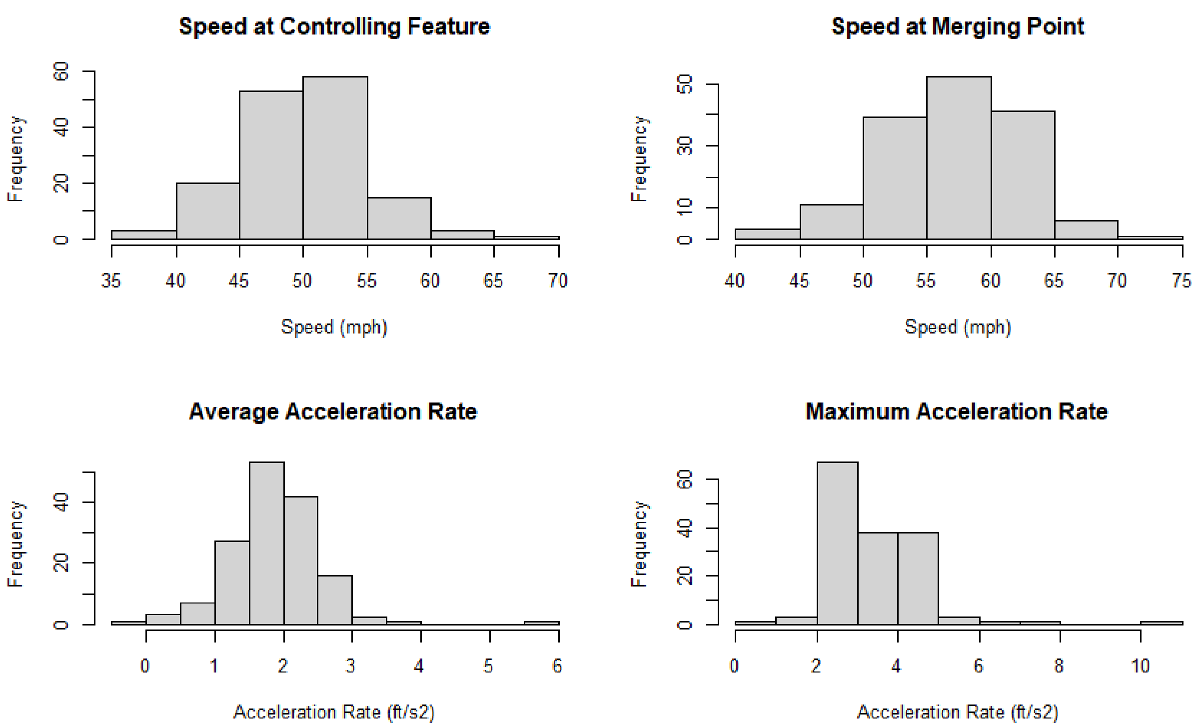


Figure 4-81. Distribution of Parameters for Site MI-5.

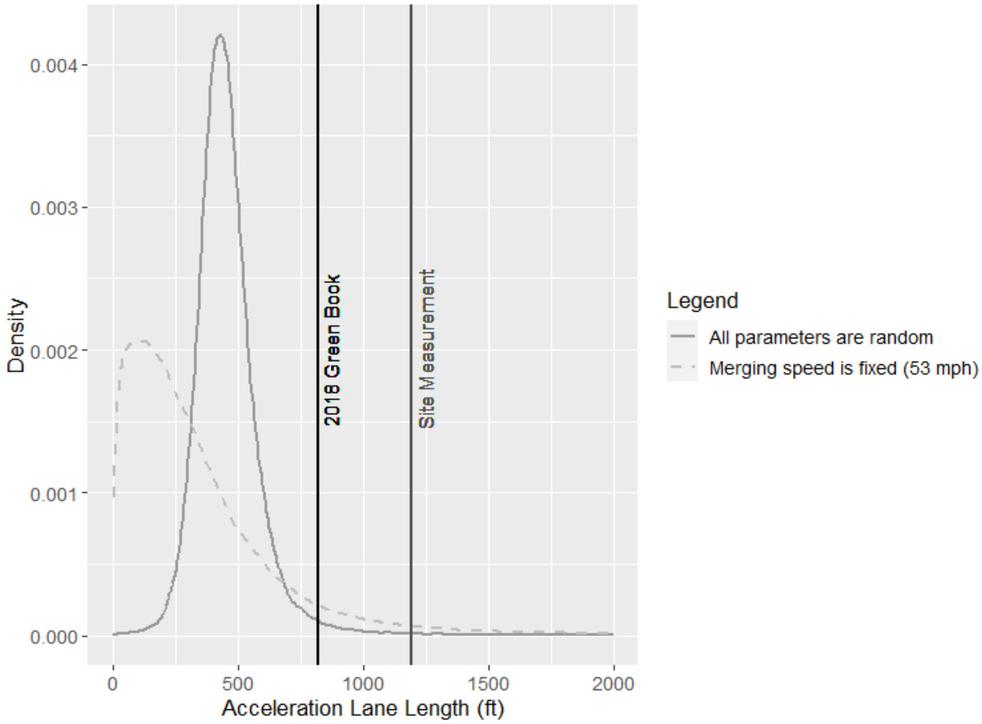


Figure 4-82. Frequency Distribution of Acceleration Lane Length based on Average Acceleration Rate for Site MI-5.

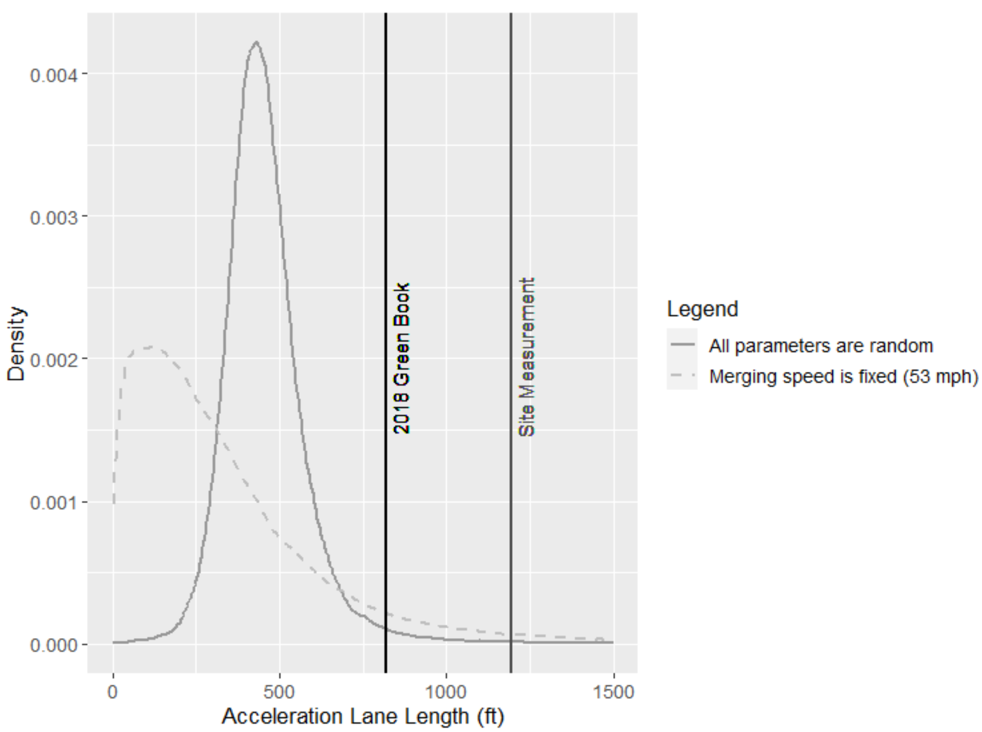


Figure 4-83. Frequency Distribution of Acceleration Lane Length based on Maximum Acceleration Rate for Site MI-5.

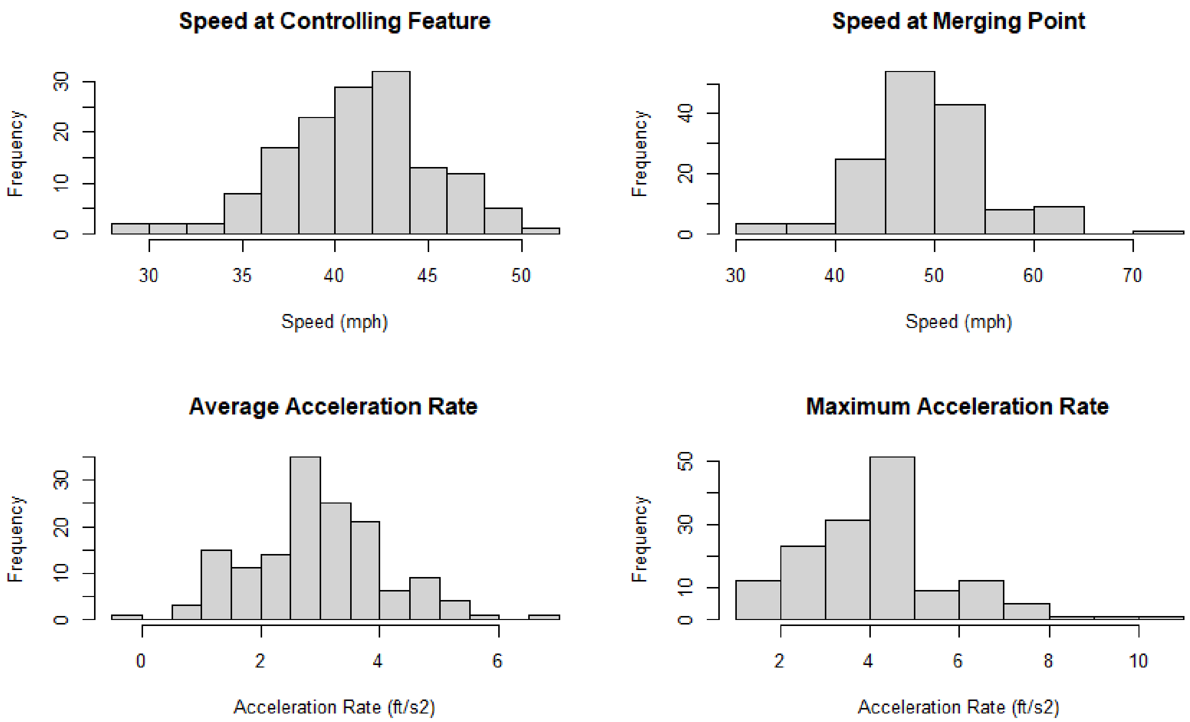


Figure 4-84. Distribution of Parameters for Site MI-7.

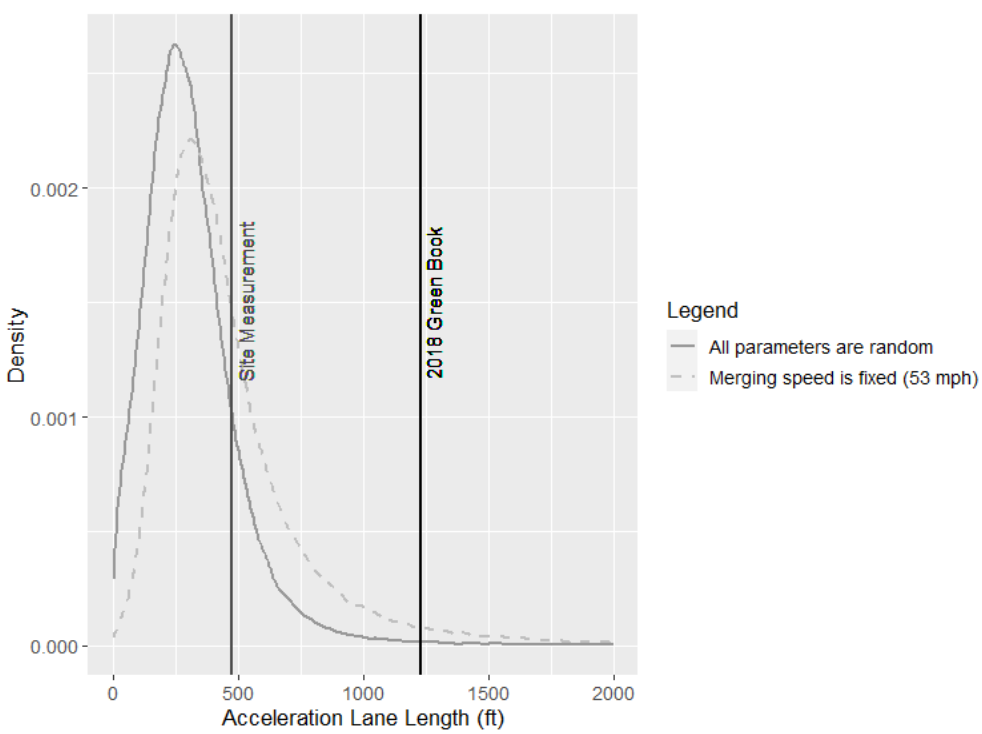


Figure 4-85. Frequency Distribution of Acceleration Lane Length based on Average Acceleration Rate for Site MI-7.

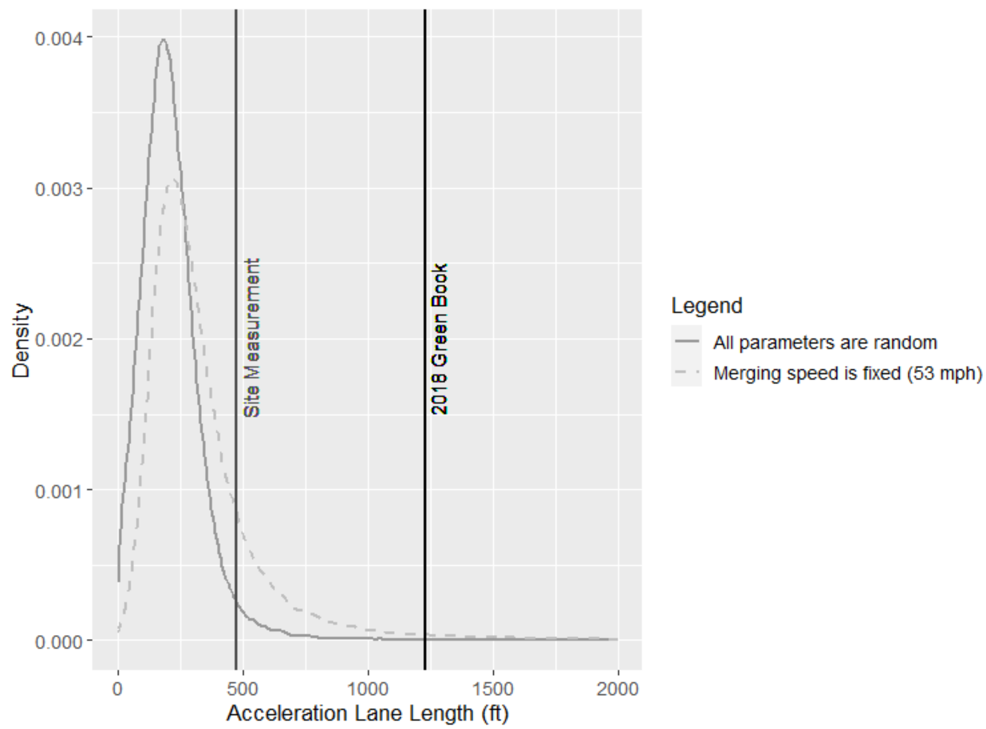


Figure 4-86. Frequency Distribution of Acceleration Lane Length based on Maximum Acceleration Rate for Site MI-7.

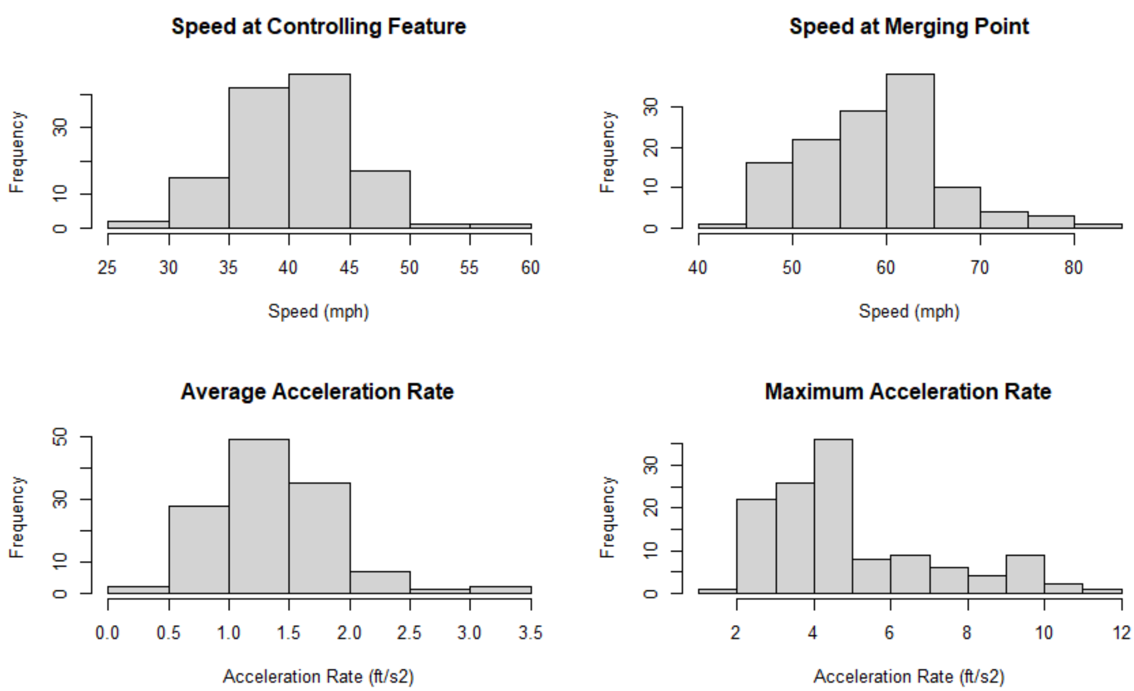


Figure 4-87. Distribution of Parameters for Site MI-8.

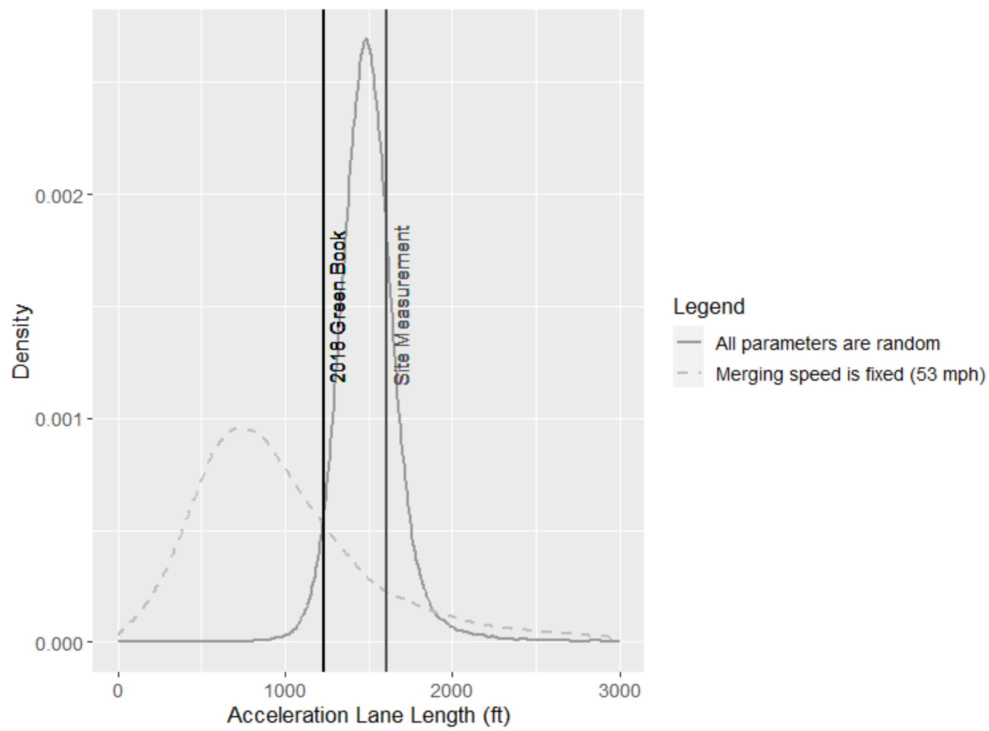


Figure 4-88. Frequency Distribution of Acceleration Lane Length based on Average Acceleration Rate for Site MI-8.

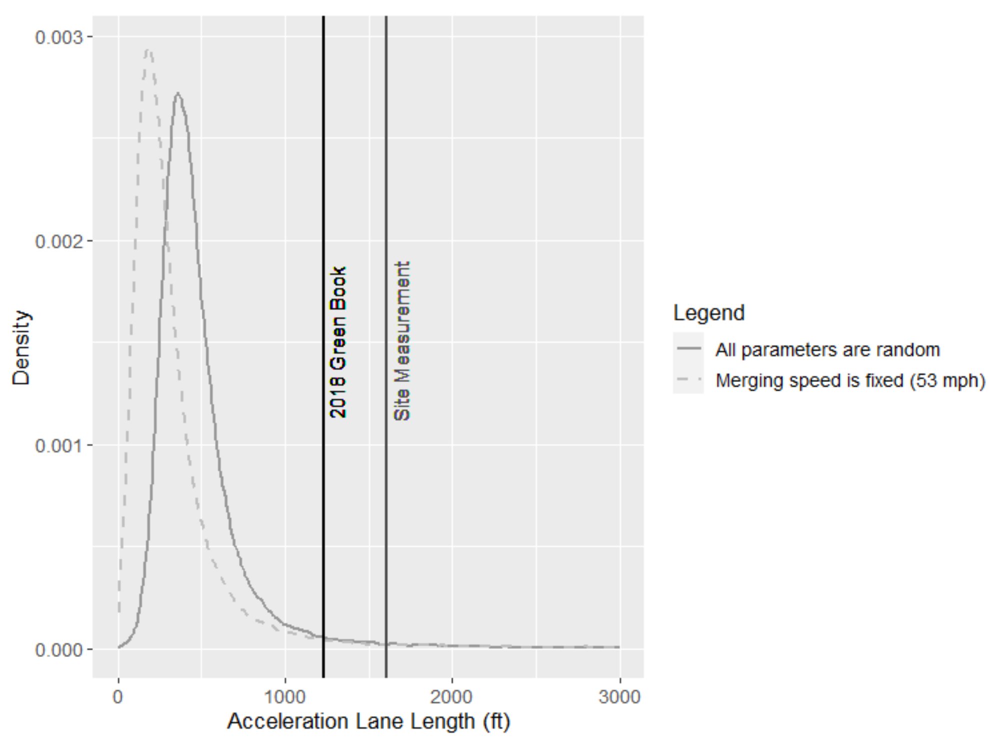


Figure 4-89. Frequency Distribution of Acceleration Lane Length based on Maximum Acceleration Rate for Site MI-8.

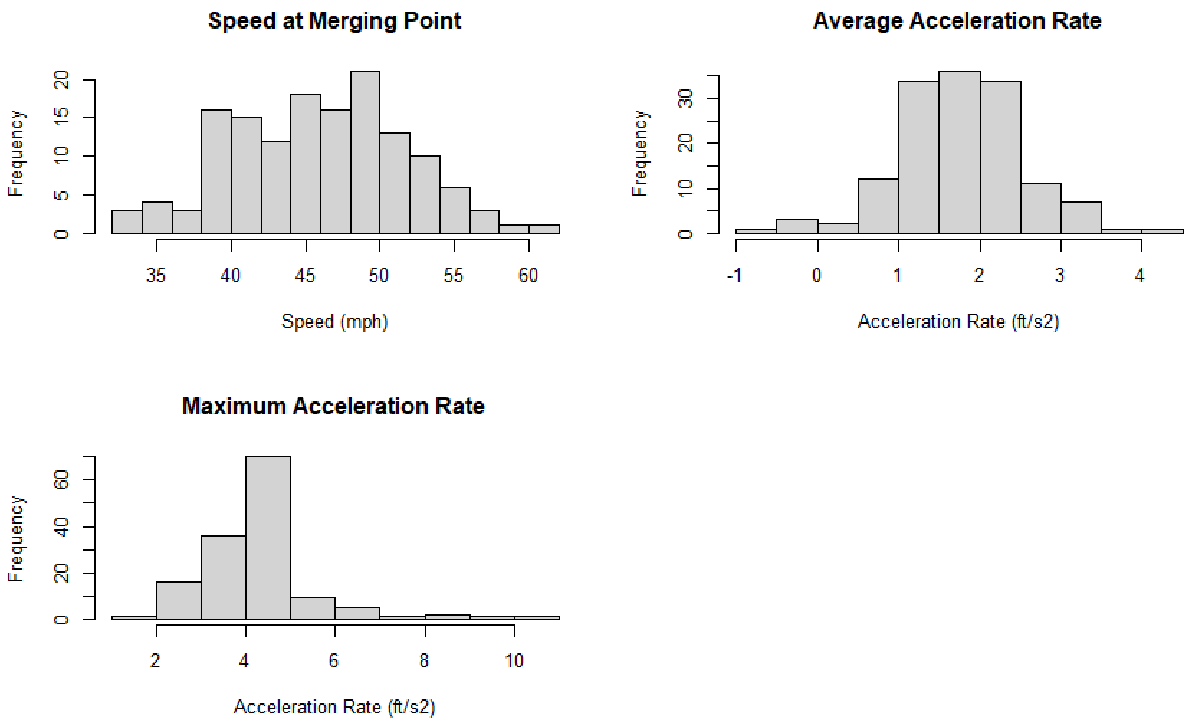


Figure 4-90. Distribution of Parameters for Site NC-1.

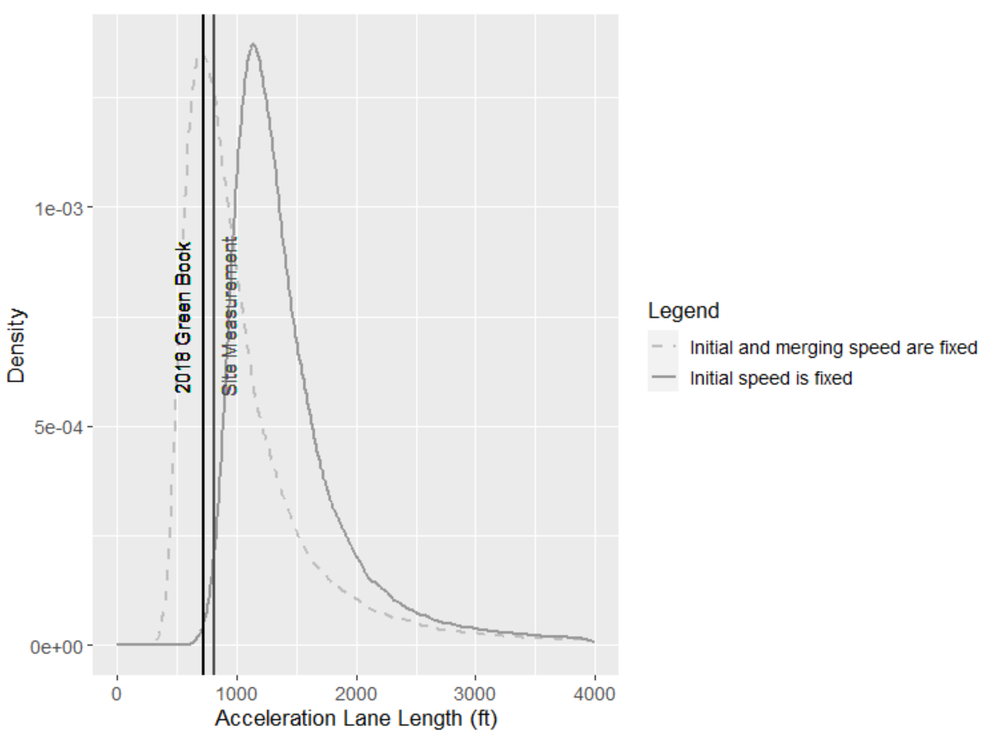


Figure 4-91. Frequency Distribution of Acceleration Lane Length based on Average Acceleration Rate for Site NC-1.

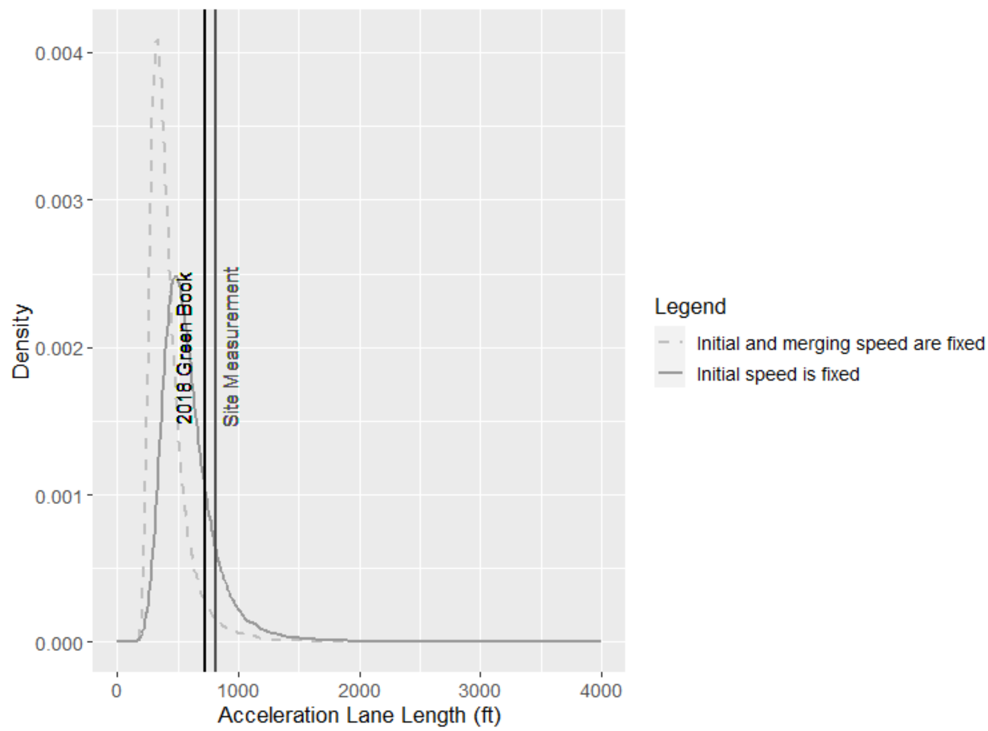


Figure 4-92. Frequency Distribution of Acceleration Lane Length based on Maximum Acceleration Rate for Site NC-1.

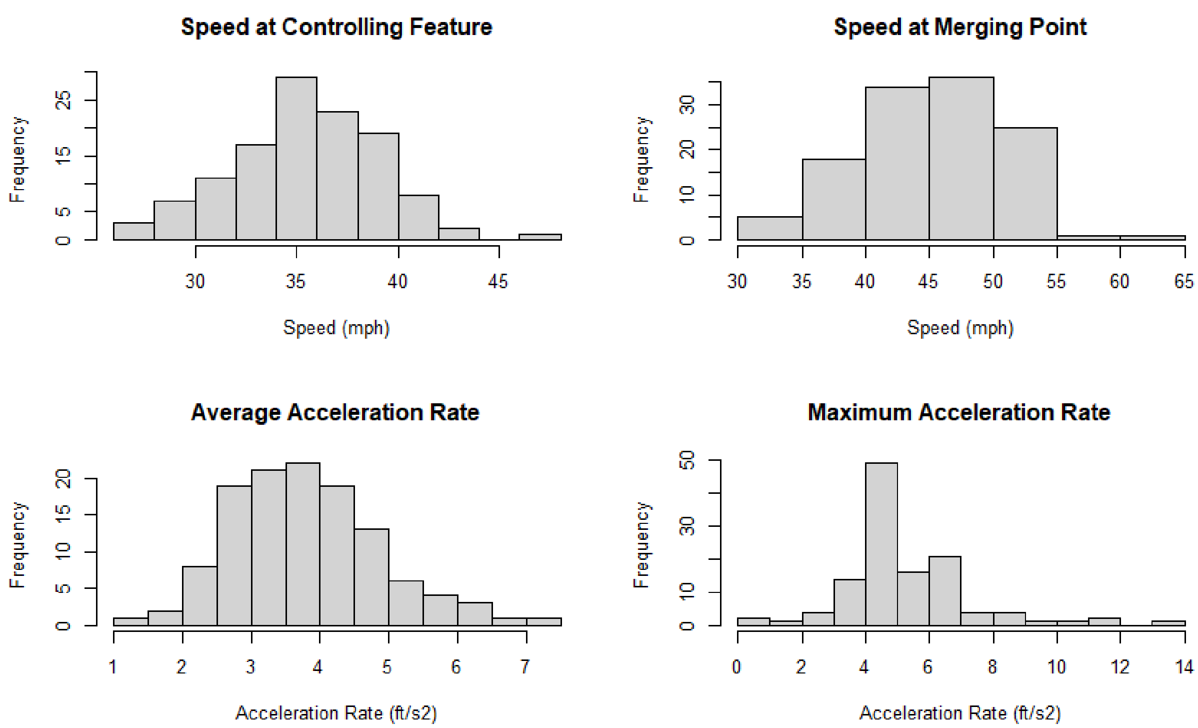


Figure 4-93. Distribution of Parameters for Site NC-2.

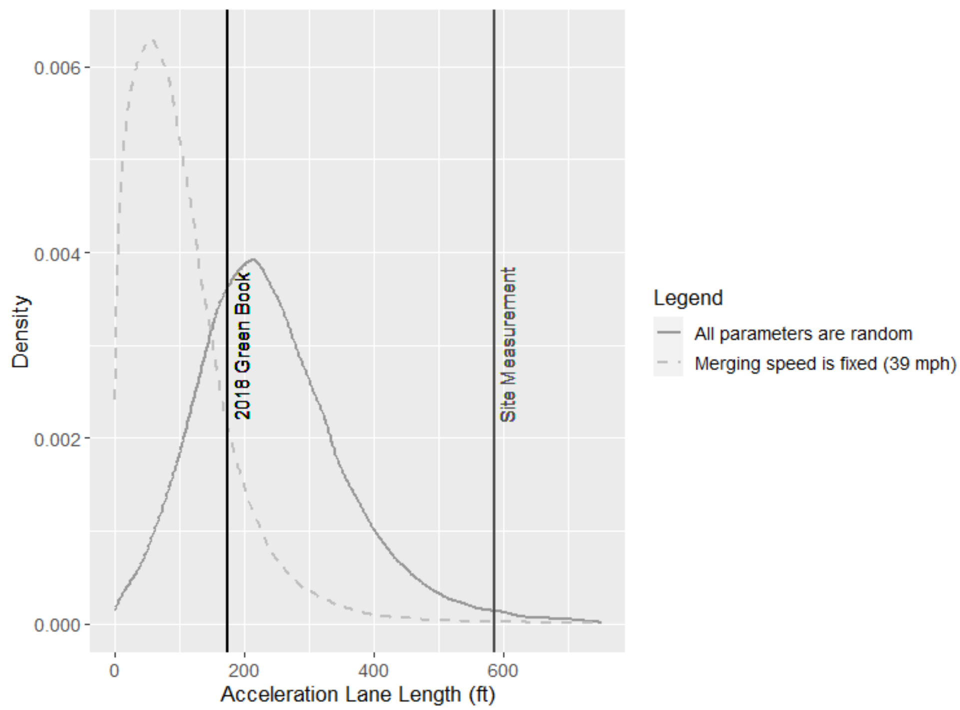


Figure 4-94. Frequency Distribution of Acceleration Lane Length based on Average Acceleration Rate for Site NC-2.

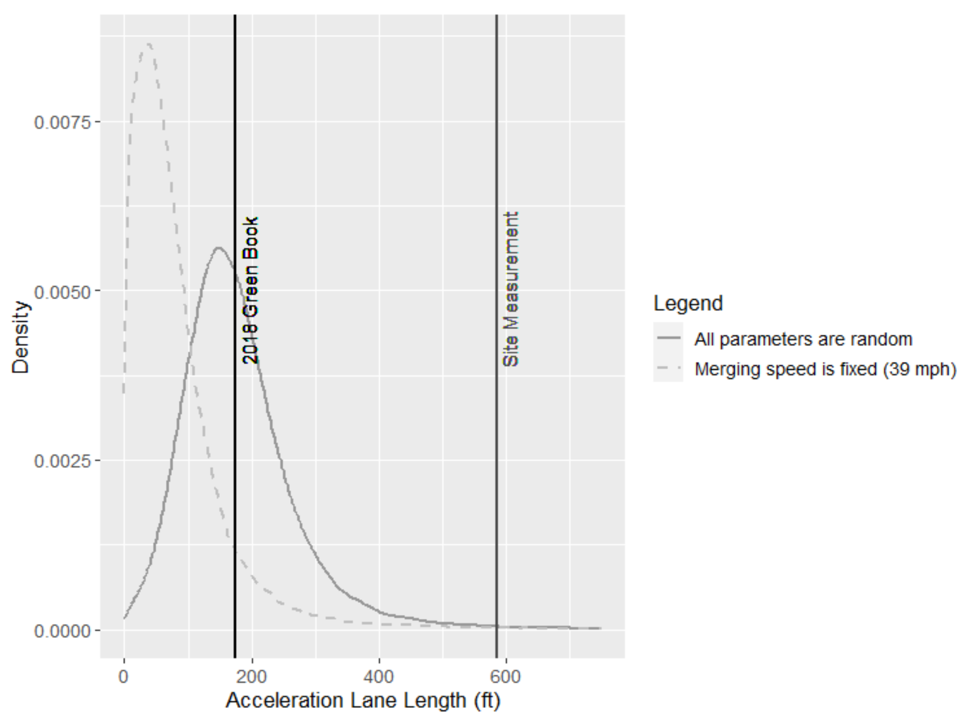


Figure 4-95. Frequency Distribution of Acceleration Lane Length based on Maximum Acceleration Rate for Site NC-2.

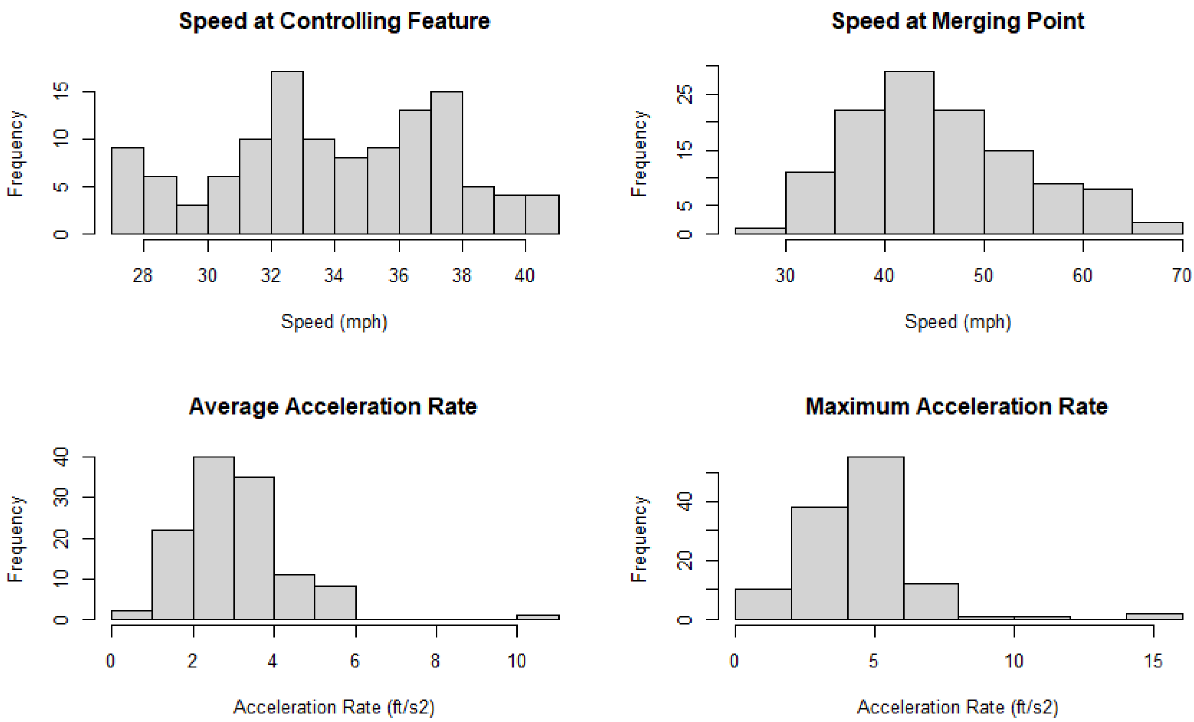


Figure 4-96. Distribution of Parameters for Site NC-3.

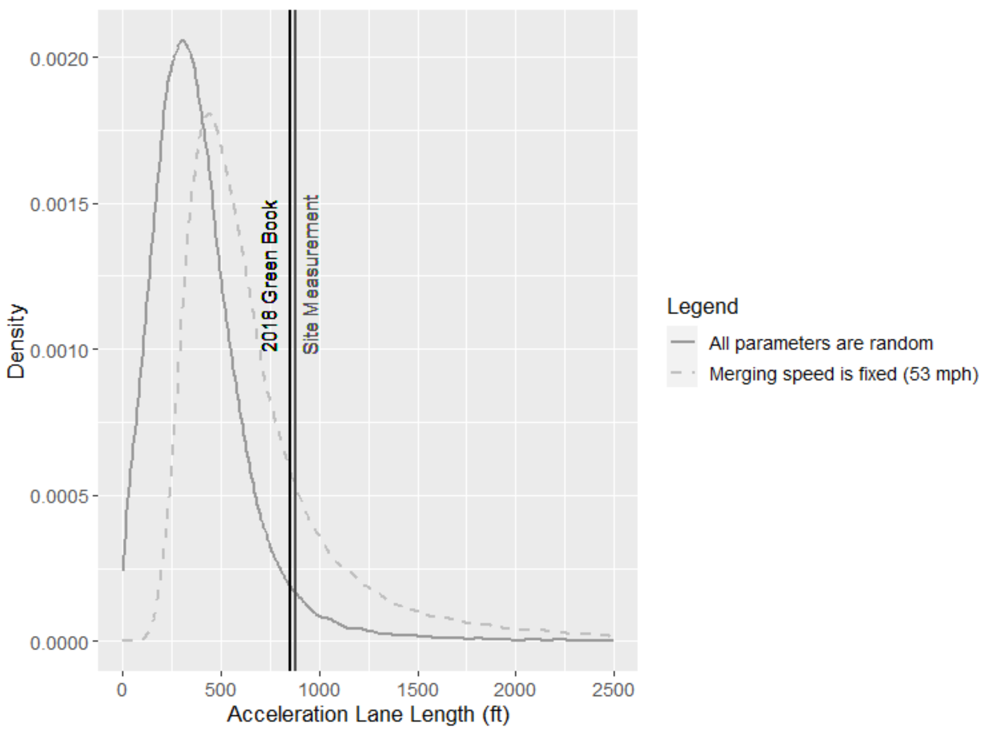


Figure 4-97. Frequency Distribution of Acceleration Lane Length based on Average Acceleration Rate for Site NC-3.

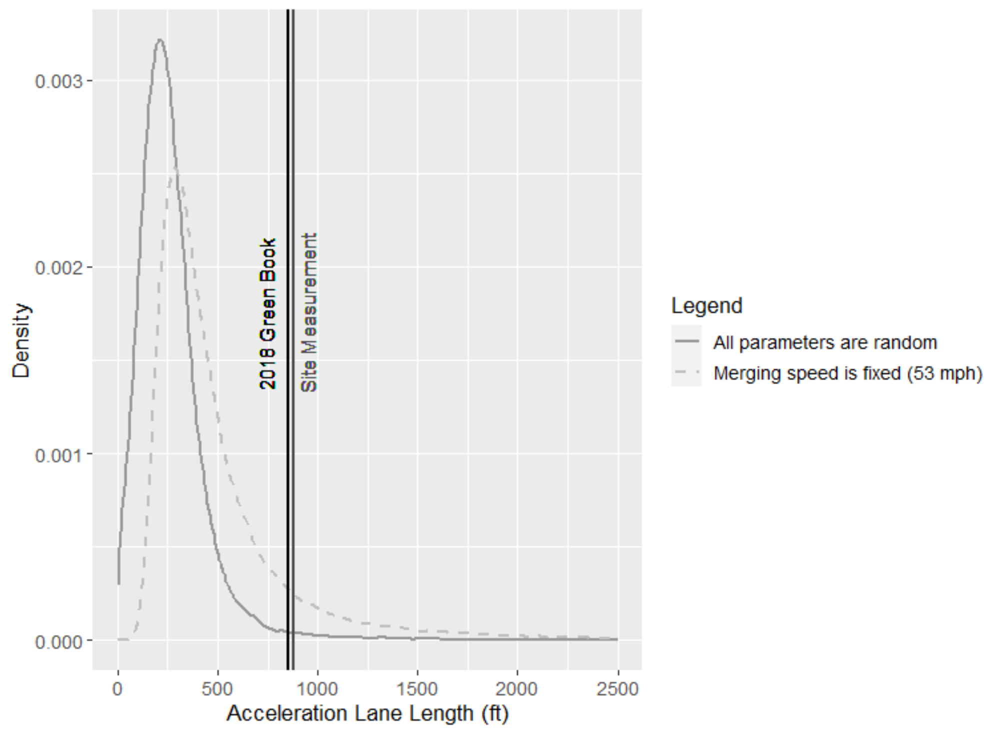


Figure 4-98. Frequency Distribution of Acceleration Lane Length based on Maximum Acceleration Rate for Site NC-3.

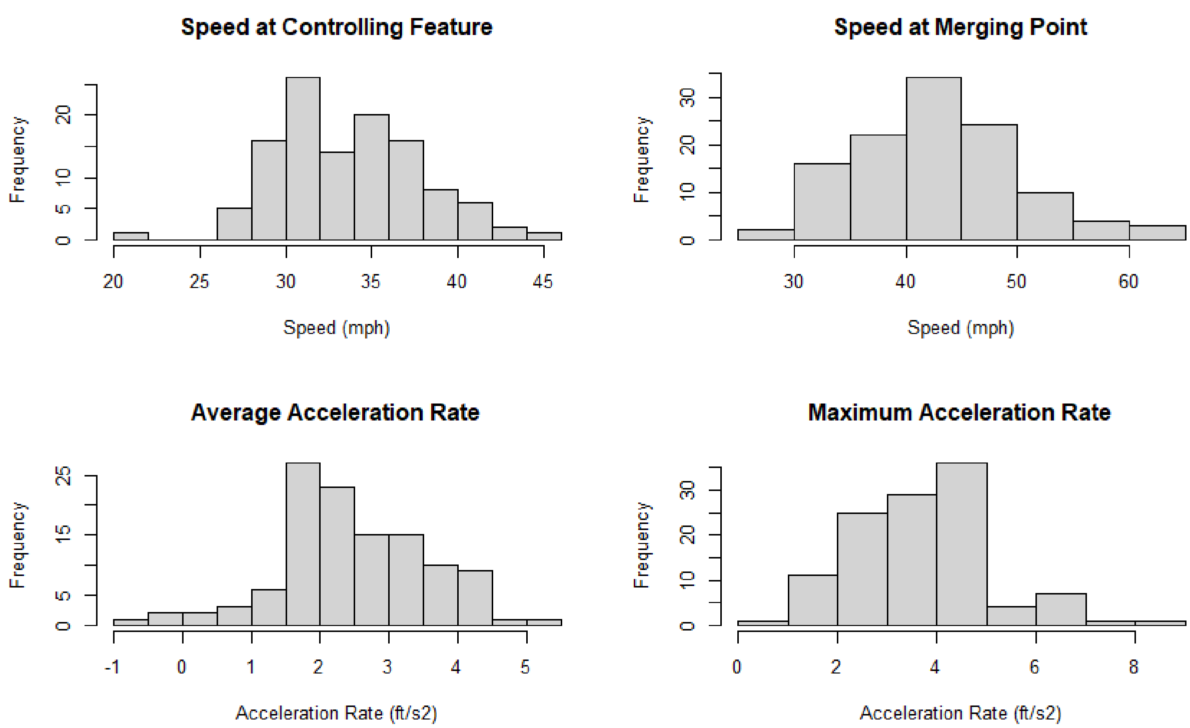


Figure 4-99. Distribution of Parameters for Site NC-4.

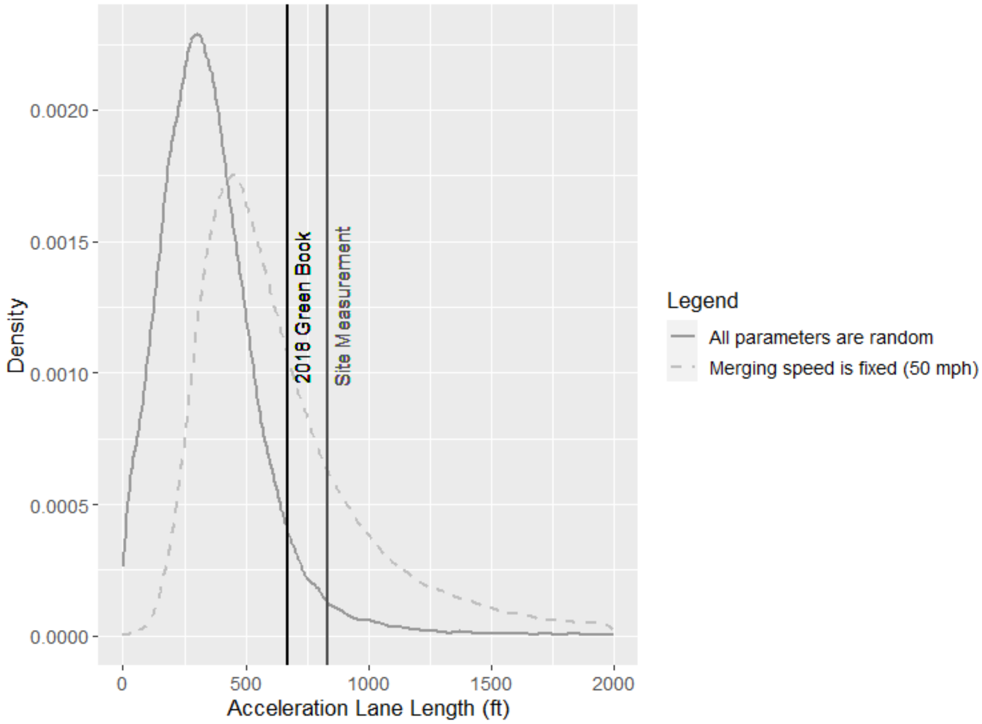


Figure 4-100. Frequency Distribution of Acceleration Lane Length based on Average Acceleration Rate for Site NC-4.

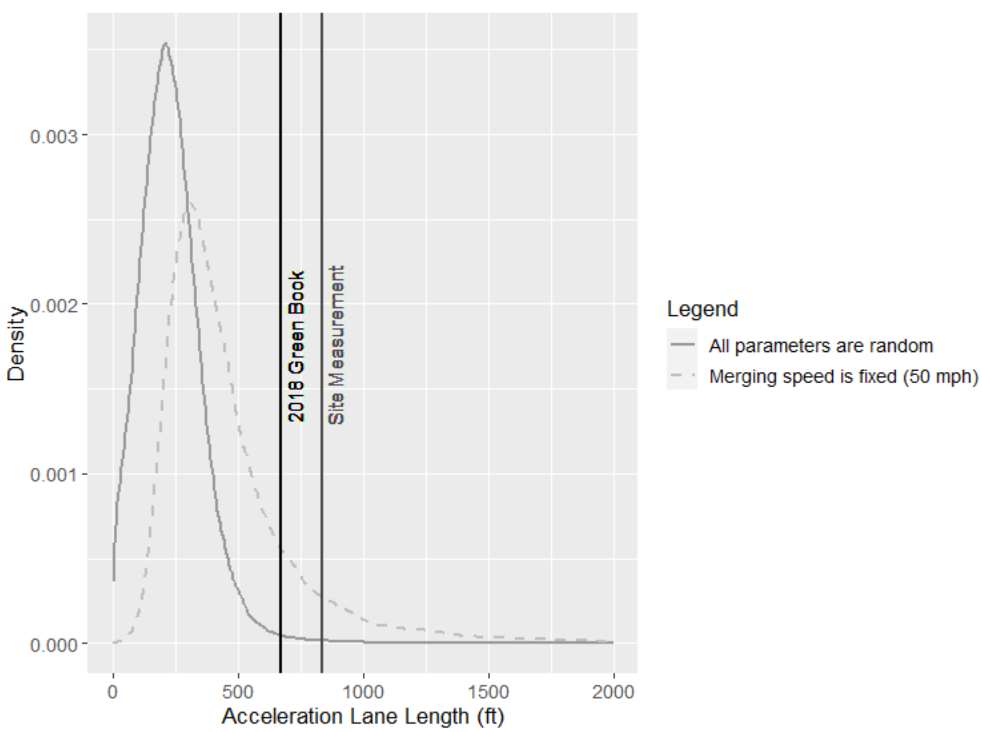


Figure 4-101. Frequency Distribution of Acceleration Lane Length based on Maximum Acceleration Rate for Site NC-4.

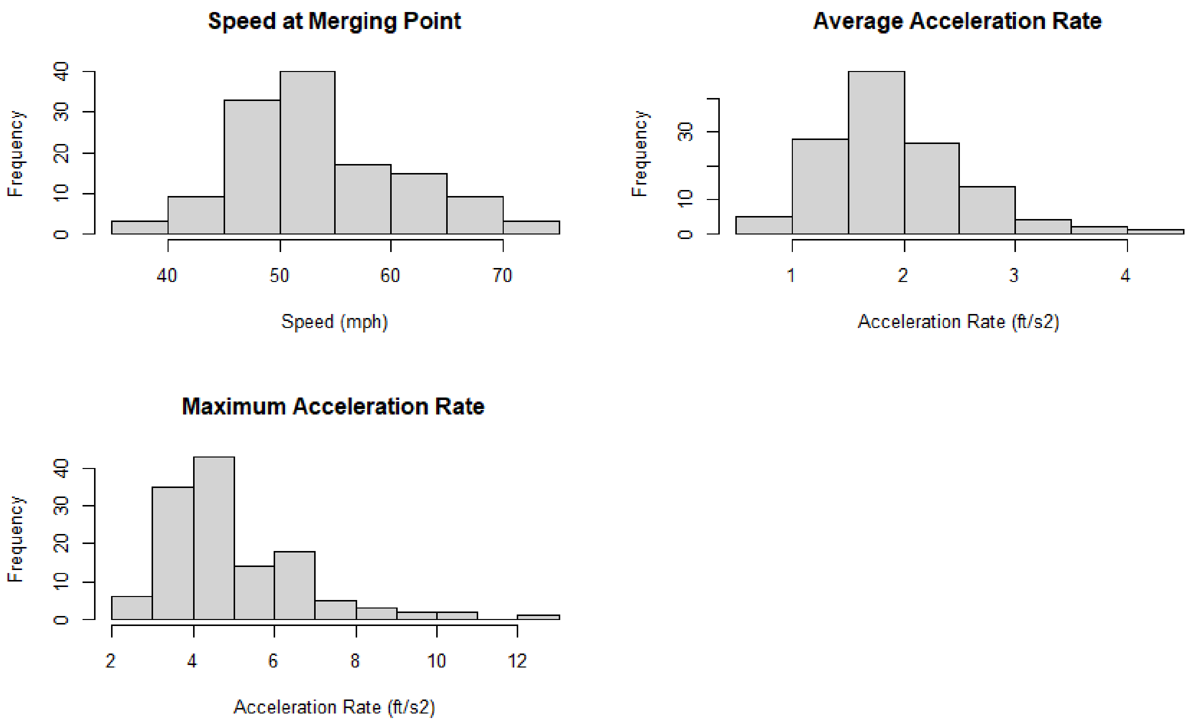


Figure 4-102. Distribution of Parameters for Site NC-5.

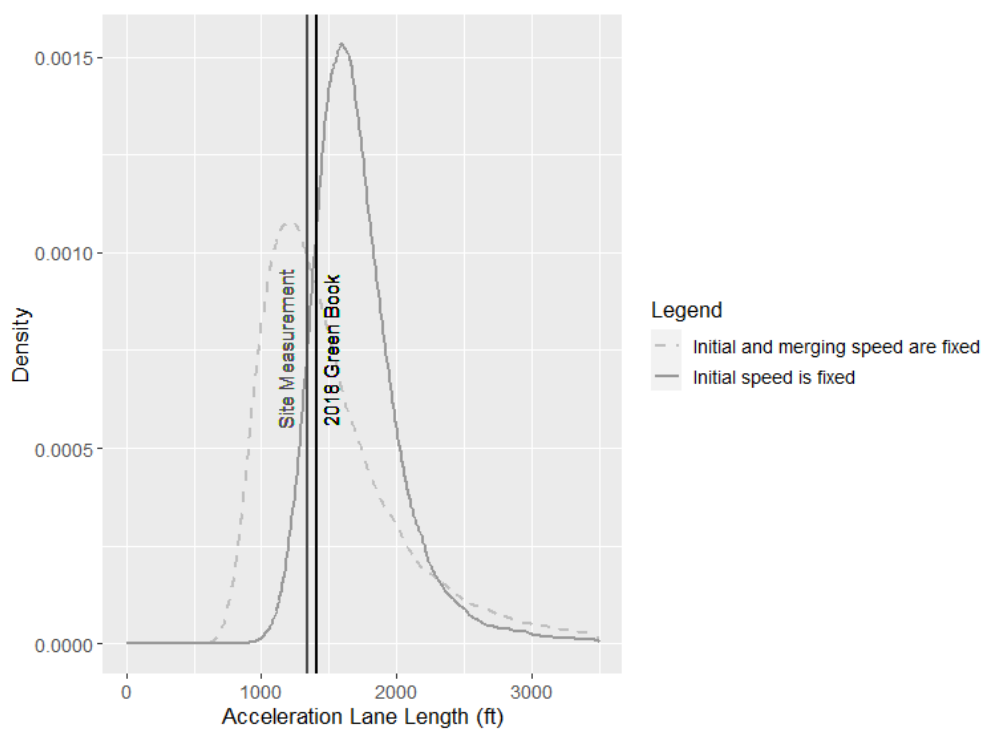


Figure 4-103. Frequency Distribution of Acceleration Lane Length based on Average Acceleration Rate for Site NC-5.

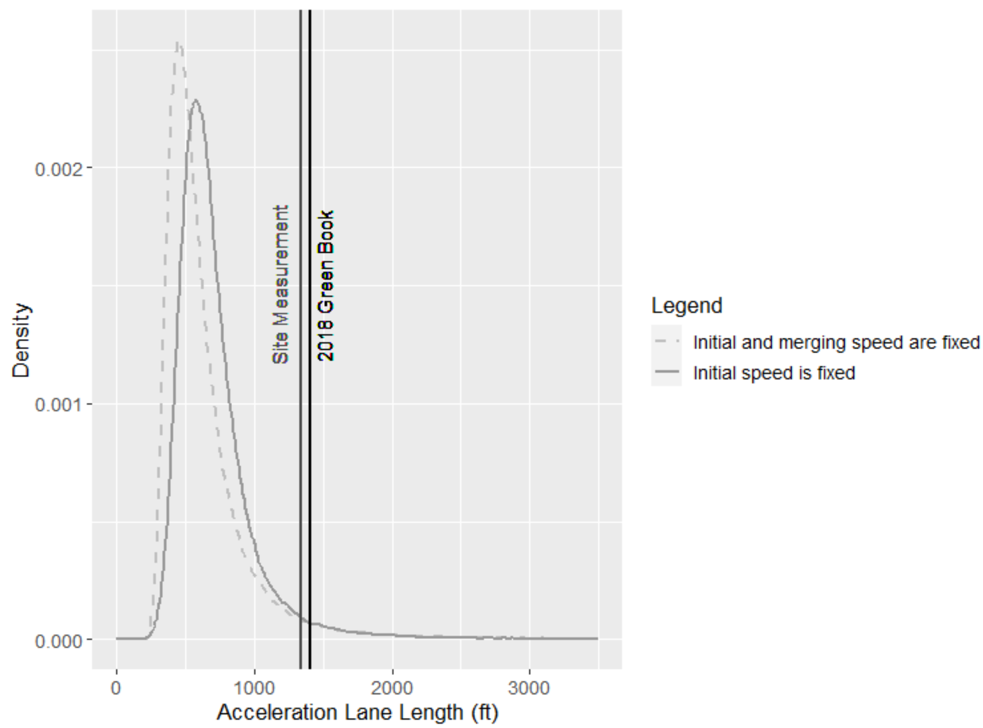


Figure 4-104. Frequency Distribution of Acceleration Lane Length based on Maximum Acceleration Rate for Site NC-5.

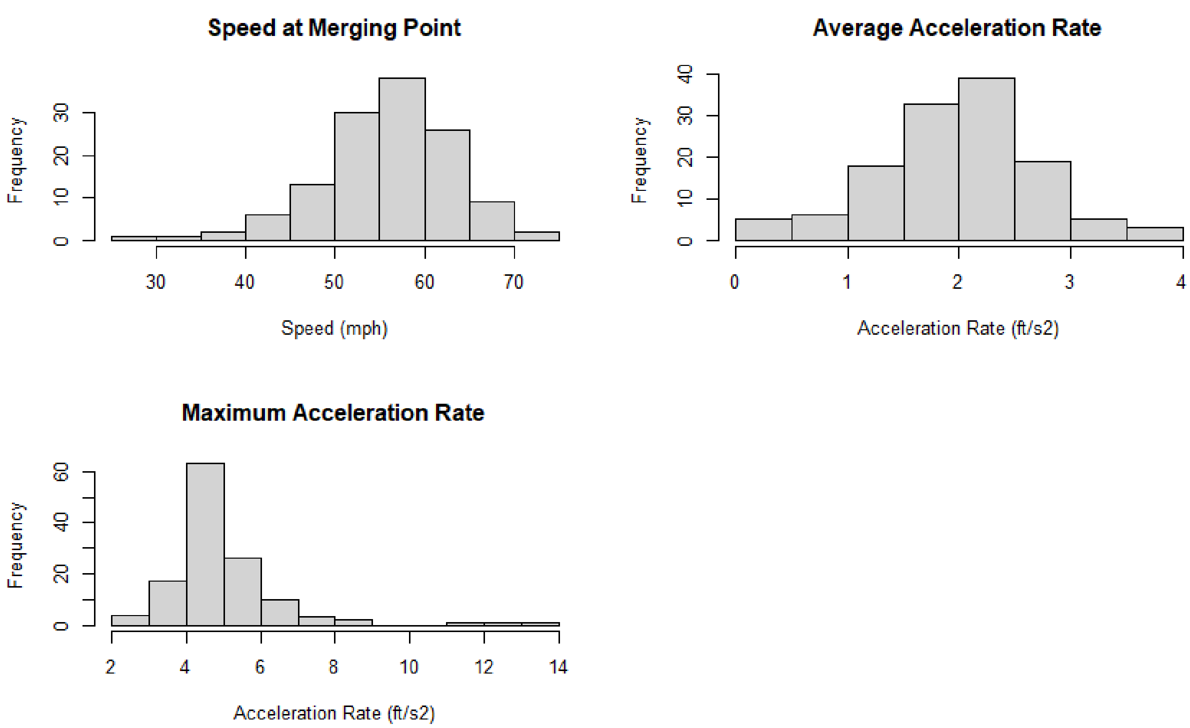


Figure 4-105. Distribution of Parameters for Site NC-6.

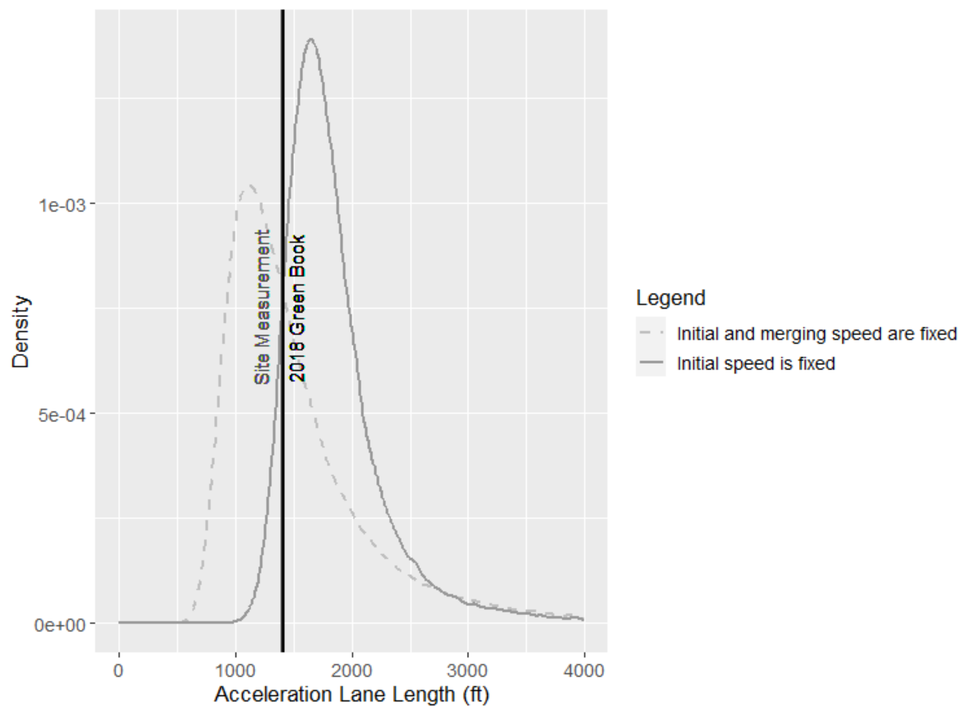


Figure 4-106. Frequency Distribution of Acceleration Lane Length based on Average Acceleration Rate for Site NC-6.

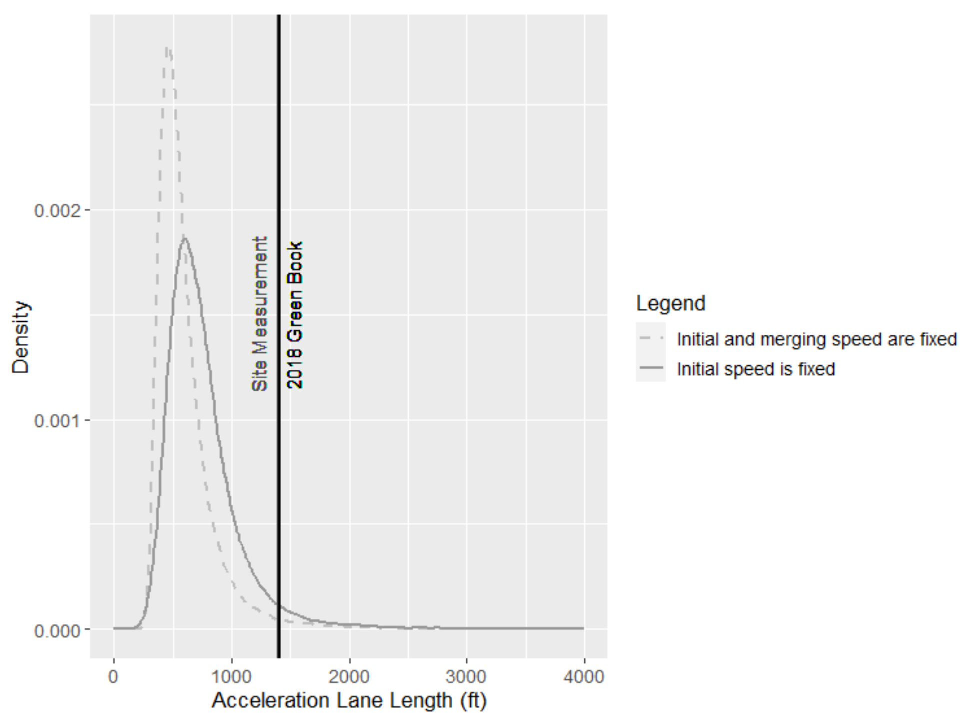


Figure 4-107. Frequency Distribution of Acceleration Lane Length based on Maximum Acceleration Rate for Site NC-6.

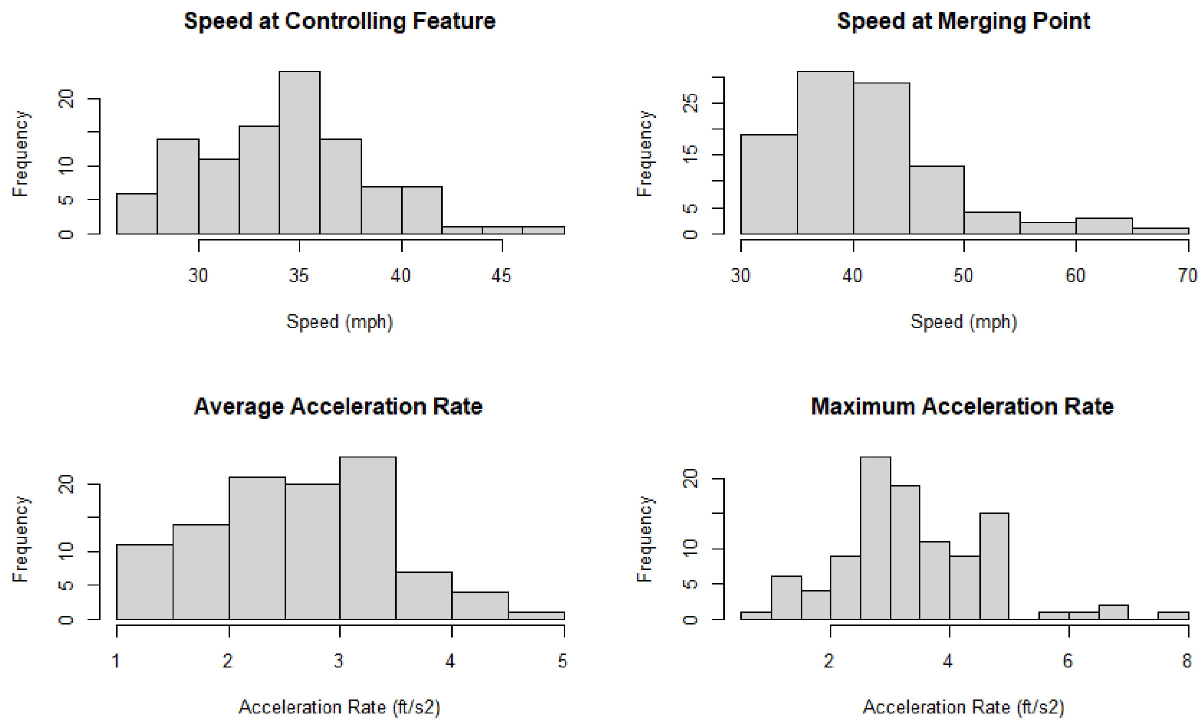


Figure 4-108. Distribution of Parameters for Site NC-7.

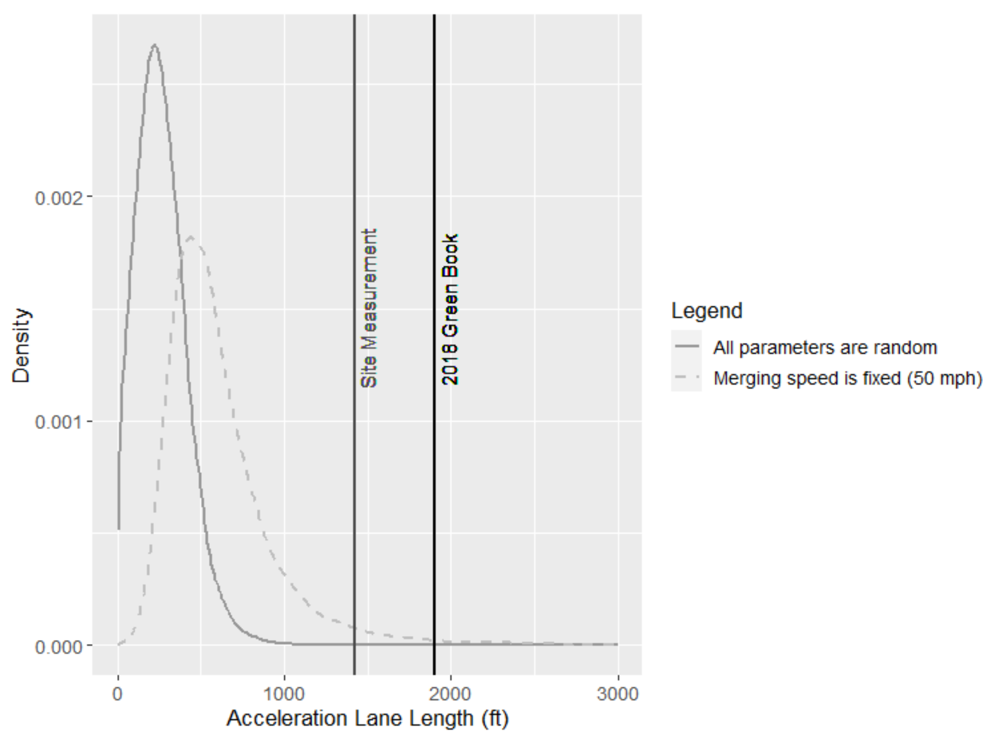


Figure 4-109. Frequency Distribution of Acceleration Lane Length based on Average Acceleration Rate for Site NC-7.

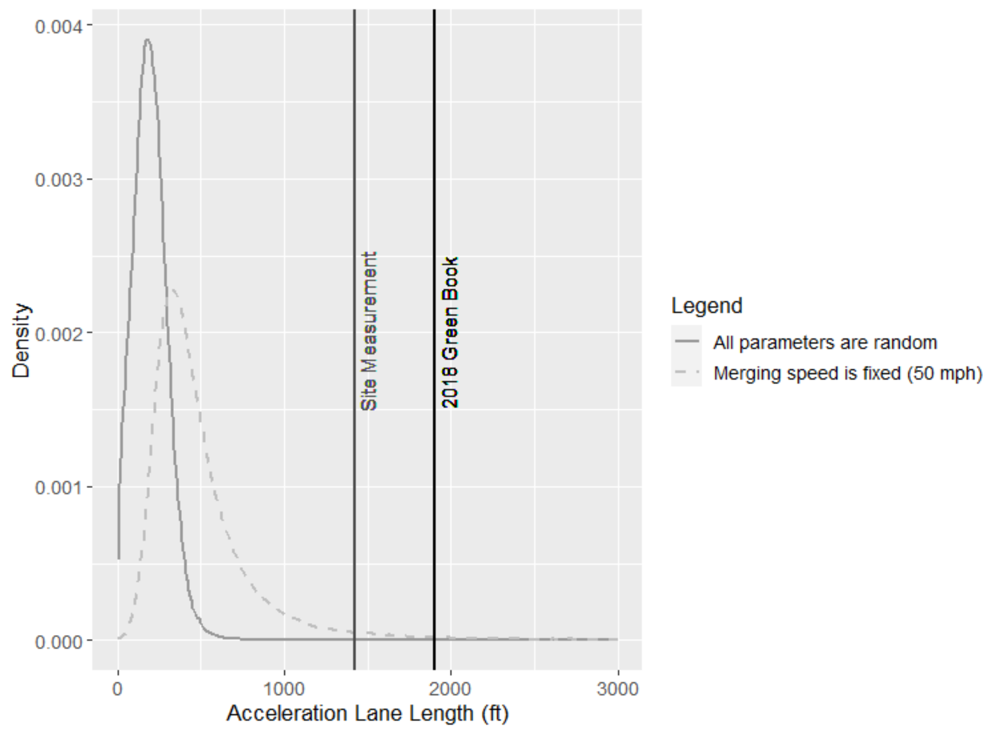


Figure 4-110. Frequency Distribution of Acceleration Lane Length based on Maximum Acceleration Rate for Site NC-7.

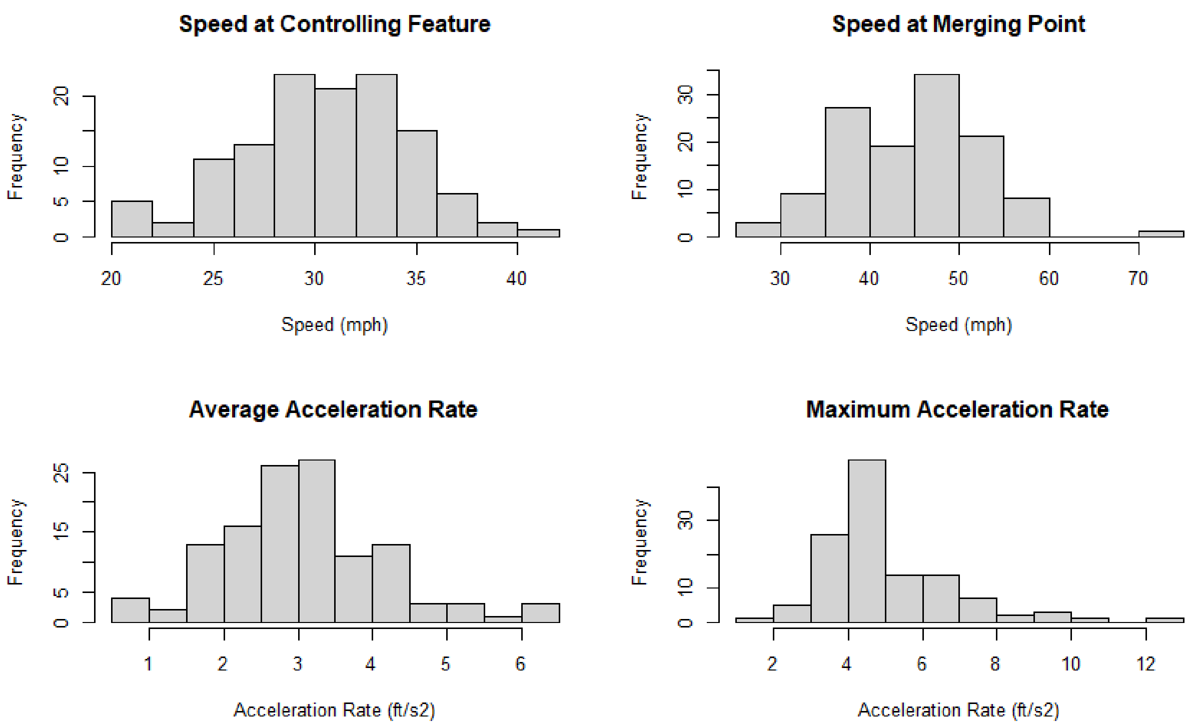


Figure 4-111. Distribution of Parameters for Site NC-8.

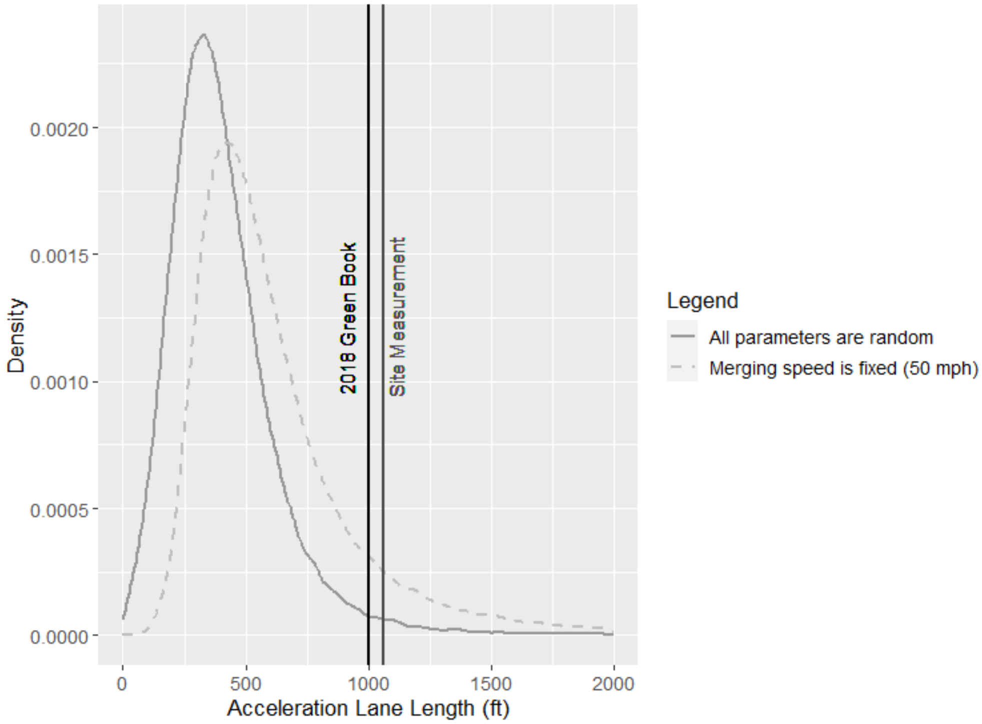


Figure 4-112. Frequency Distribution of Acceleration Lane Length based on Average Acceleration Rate for Site NC-8.

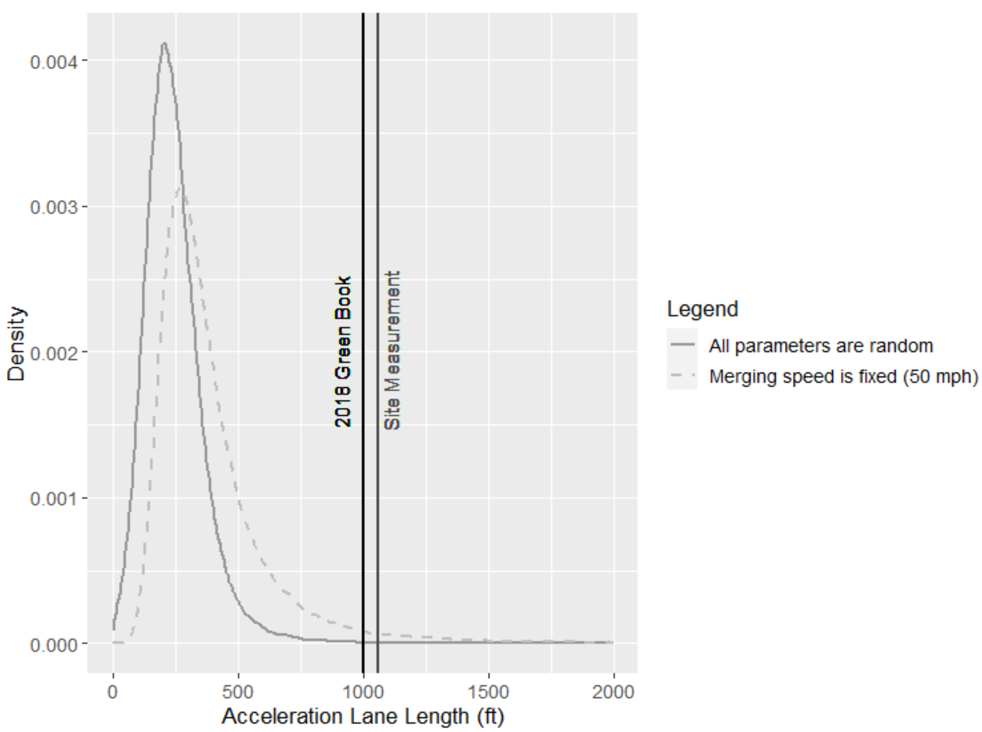


Figure 4-113. Frequency Distribution of Acceleration Lane Length based on Maximum Acceleration Rate for Site NC-8.

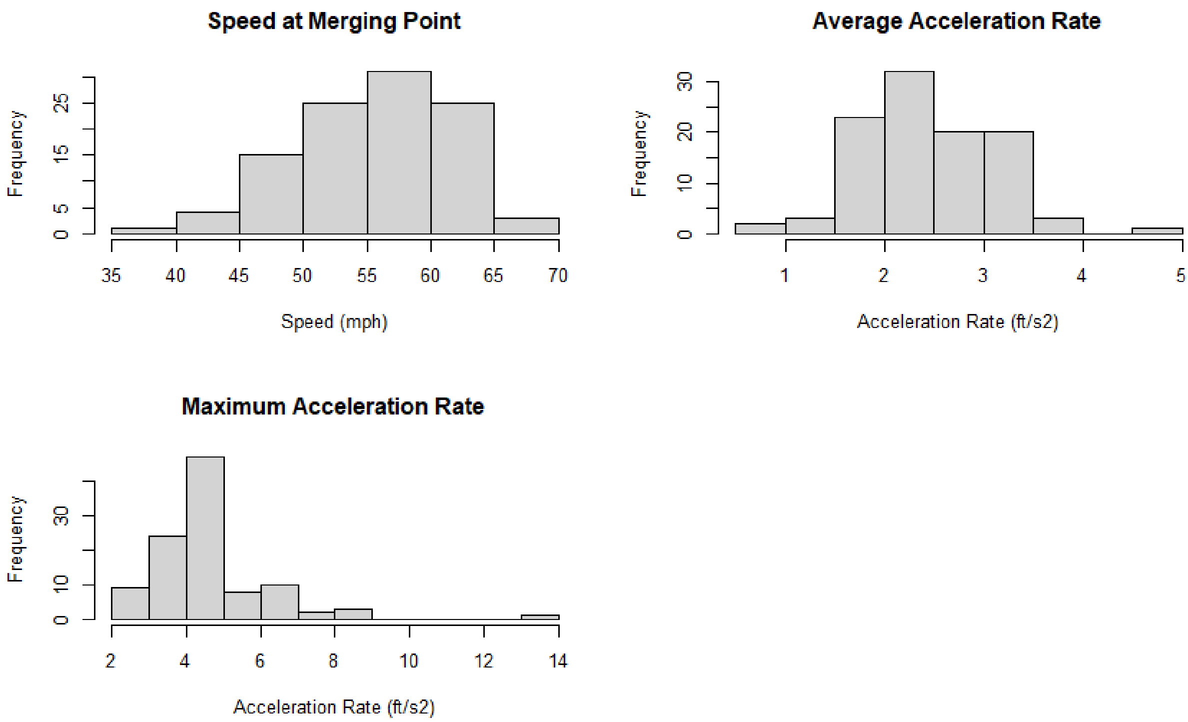


Figure 4-114. Distribution of Parameters for Site PA-1.

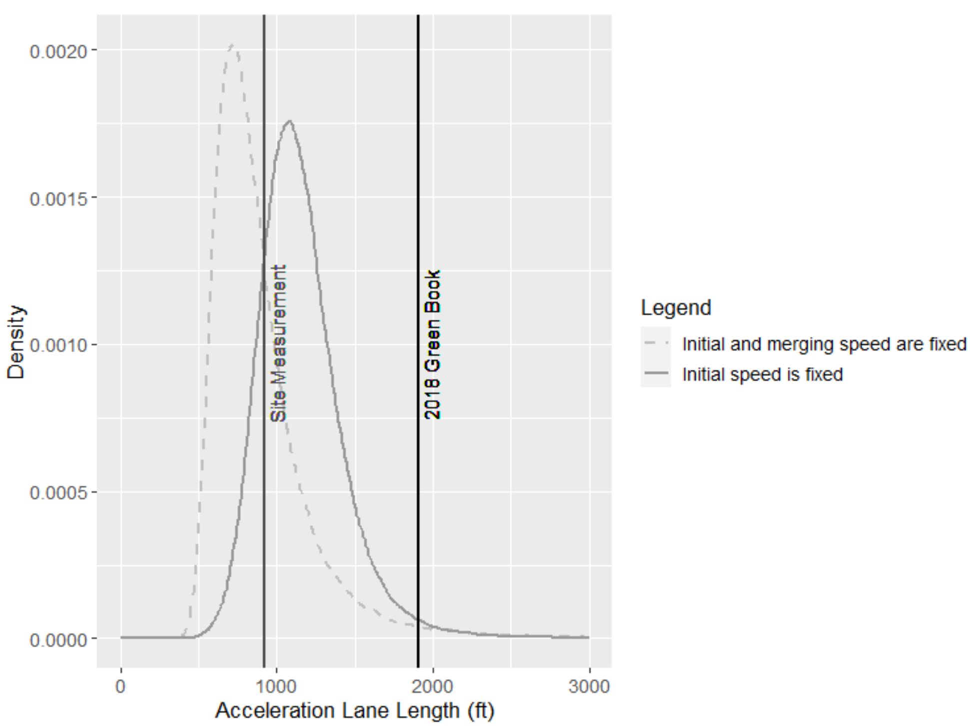


Figure 4-115. Frequency Distribution of Acceleration Lane Length based on Average Acceleration Rate for Site PA-1.

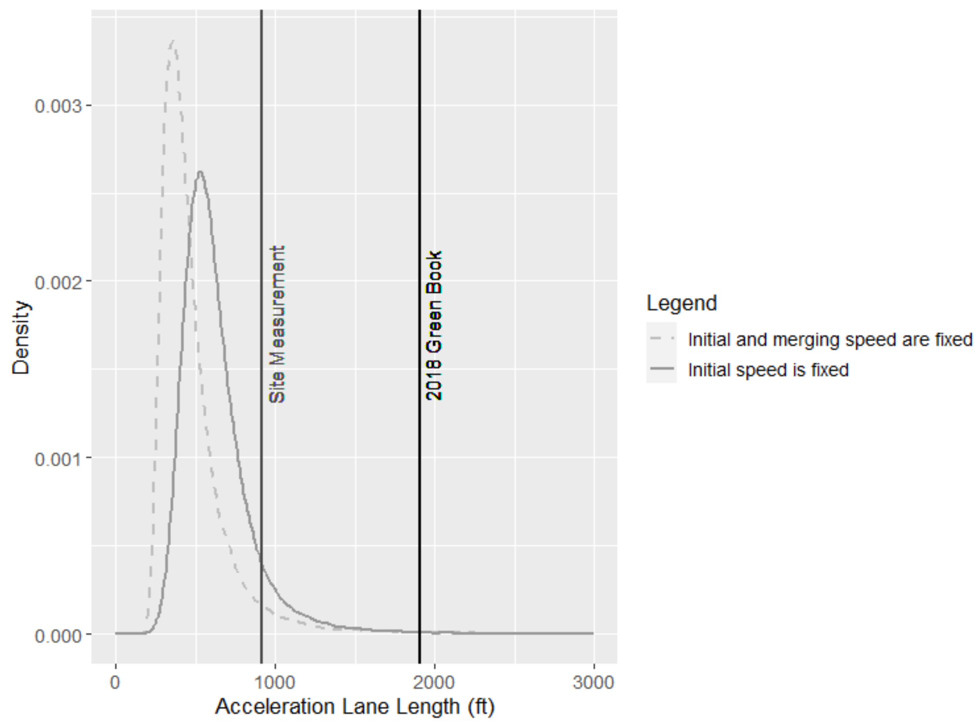


Figure 4-116. Frequency Distribution of Acceleration Lane Length based on Maximum Acceleration Rate for Site PA-1.

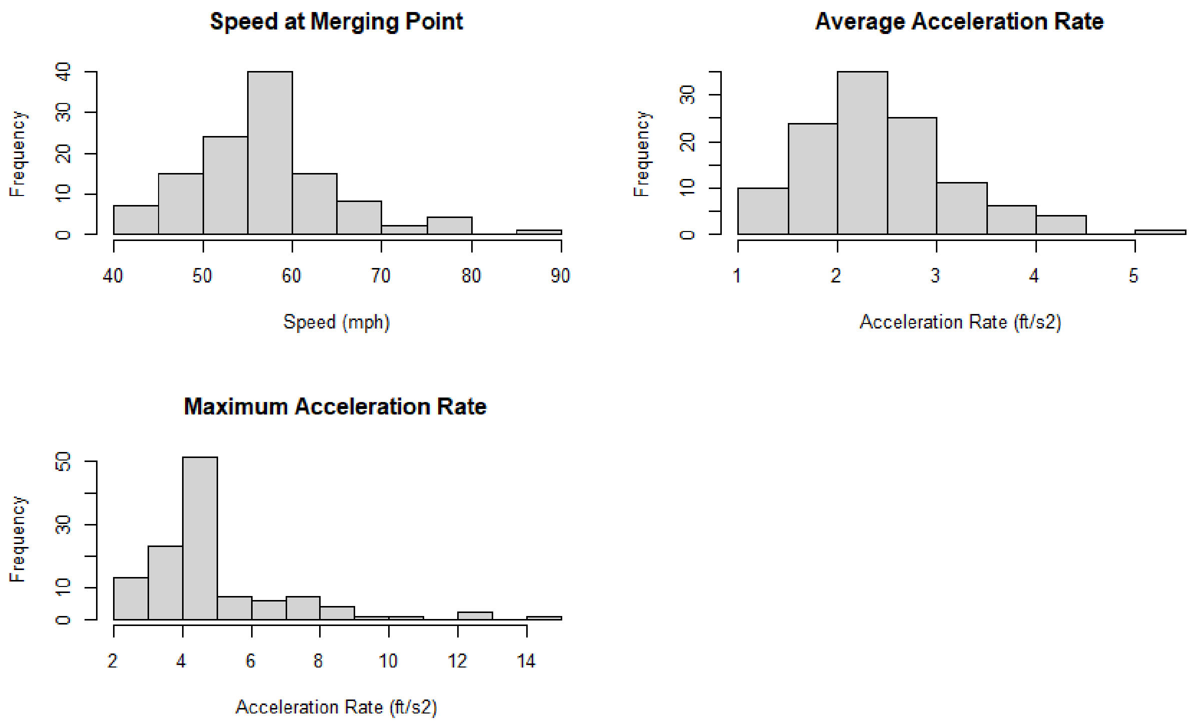


Figure 4-117. Distribution of Parameters for Site PA-2.

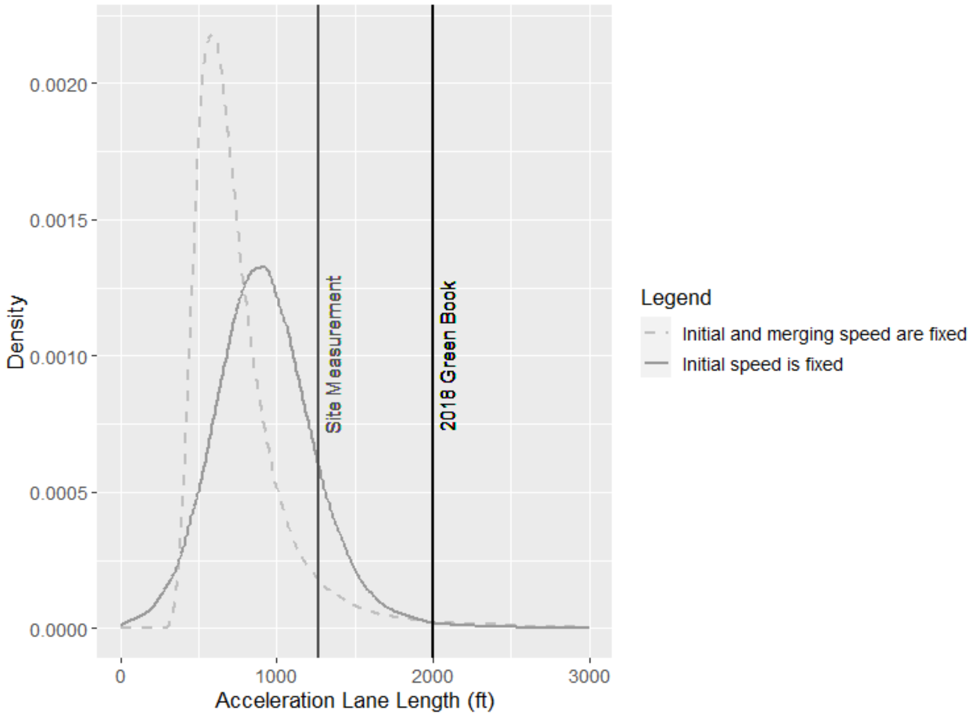


Figure 4-118. Frequency Distribution of Acceleration Lane Length based on Average Acceleration Rate for Site PA-2.

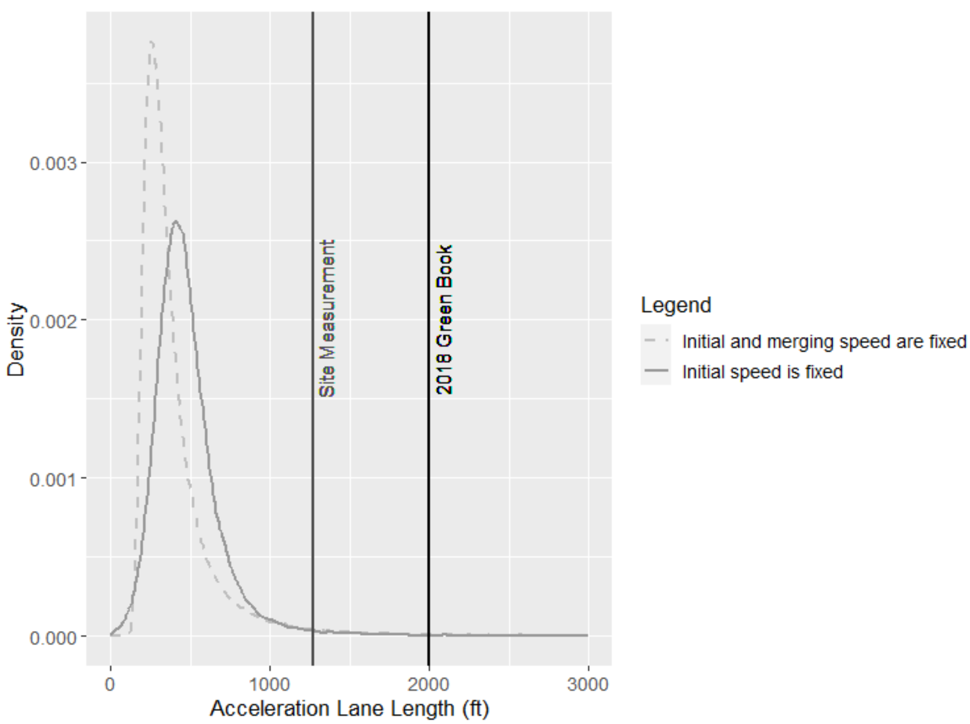


Figure 4-119. Frequency Distribution of Acceleration Lane Length based on Maximum Acceleration Rate for Site PA-2.

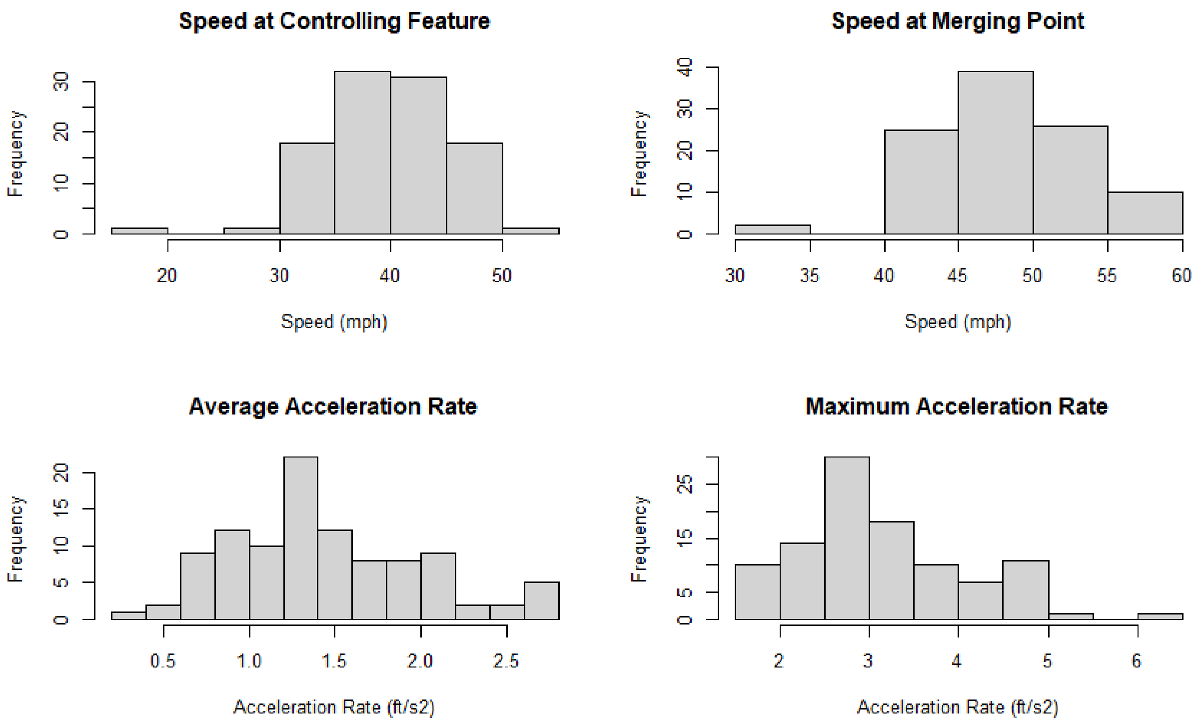


Figure 4-120. Distribution of Parameters for Site PA-3.

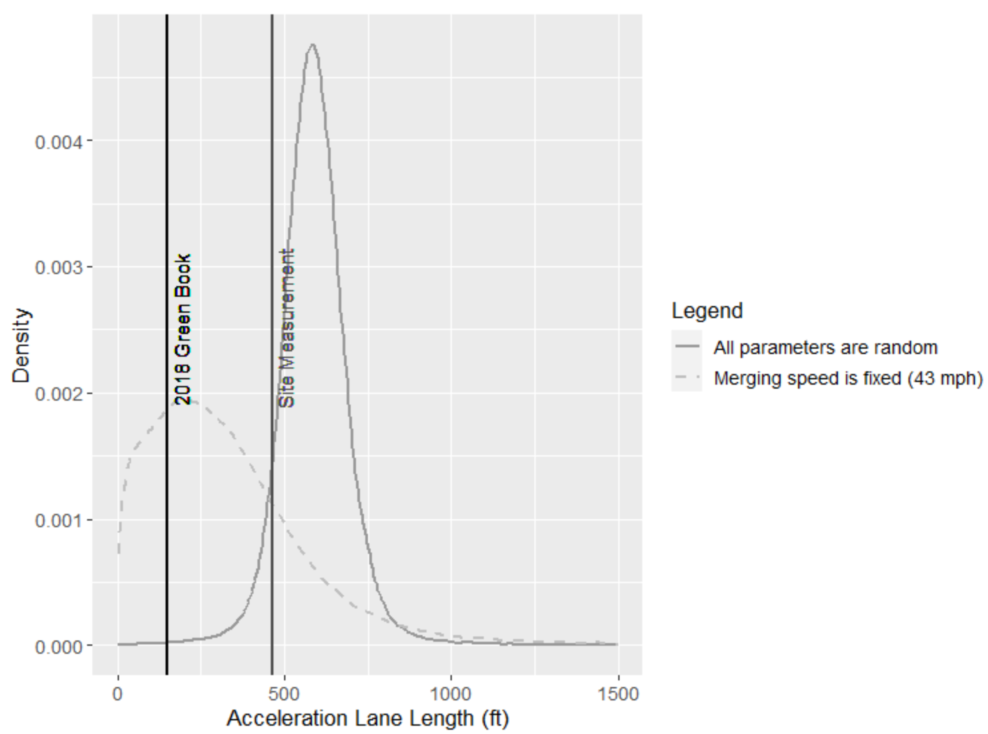


Figure 4-121. Frequency Distribution of Acceleration Lane Length based on Average Acceleration Rate for Site PA-3.

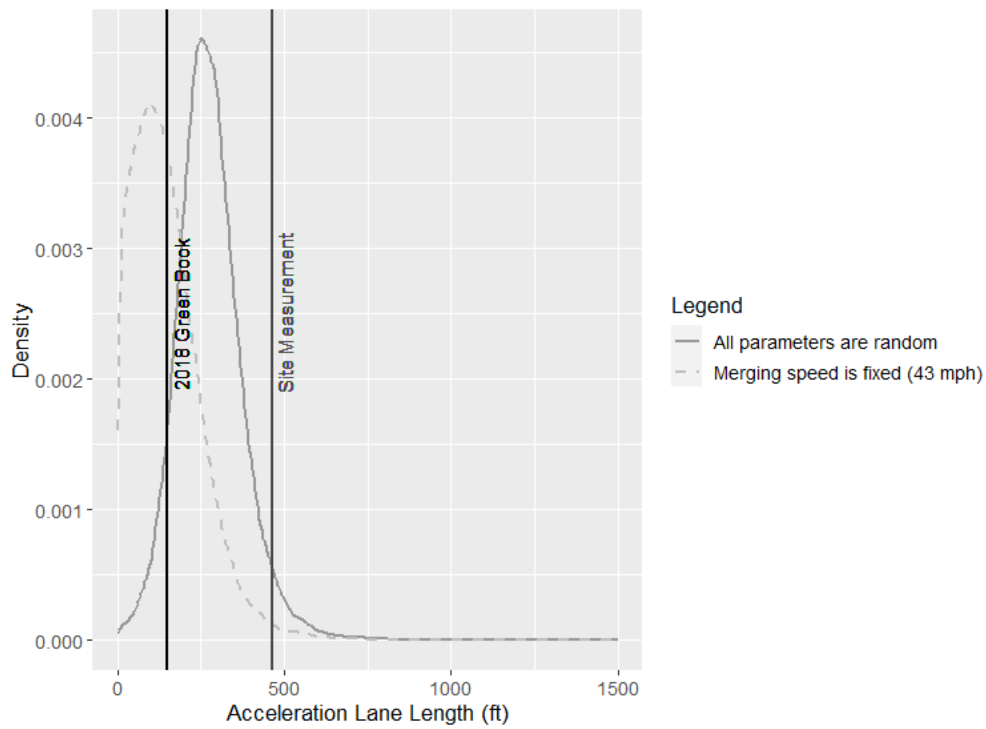


Figure 4-122. Frequency Distribution of Acceleration Lane Length based on Maximum Acceleration Rate for Site PA-3.

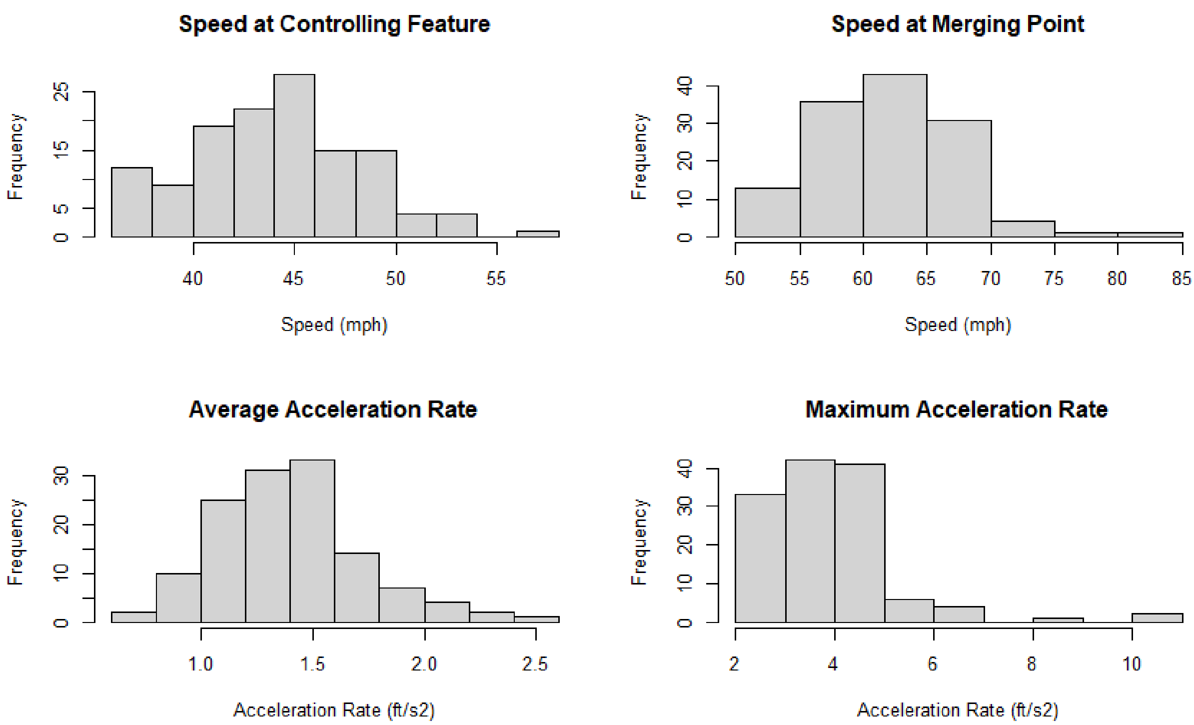


Figure 4-123. Distribution of Parameters for Site PA-4.

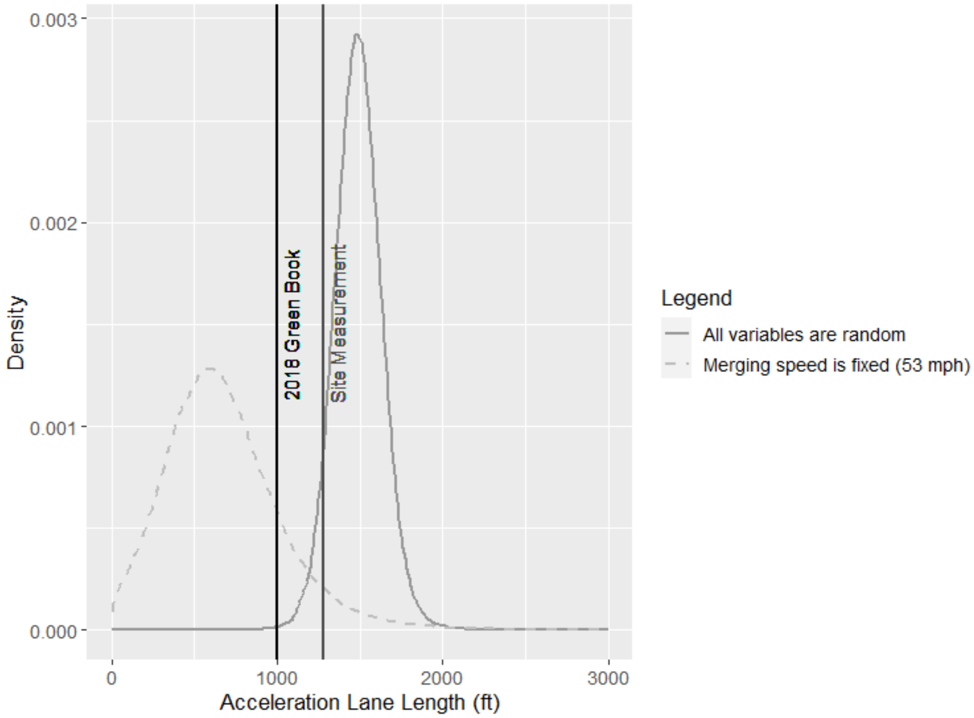


Figure 4-124. Frequency Distribution of Acceleration Lane Length based on Average Acceleration Rate for Site PA-4.

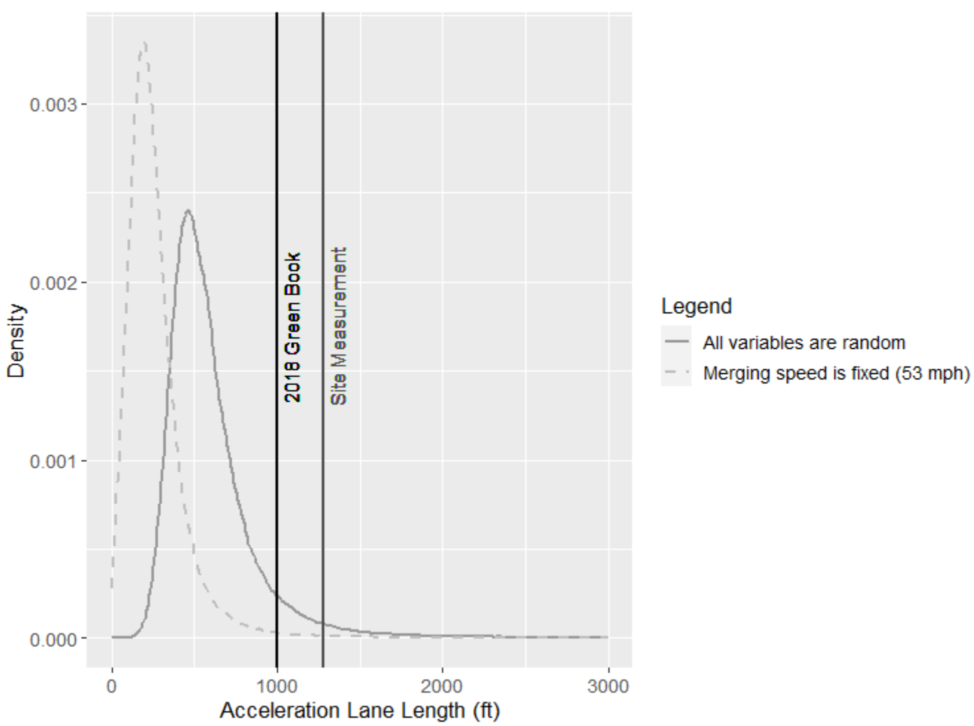


Figure 4-125. Frequency Distribution of Acceleration Lane Length based on Maximum Acceleration Rate for Site PA-4.

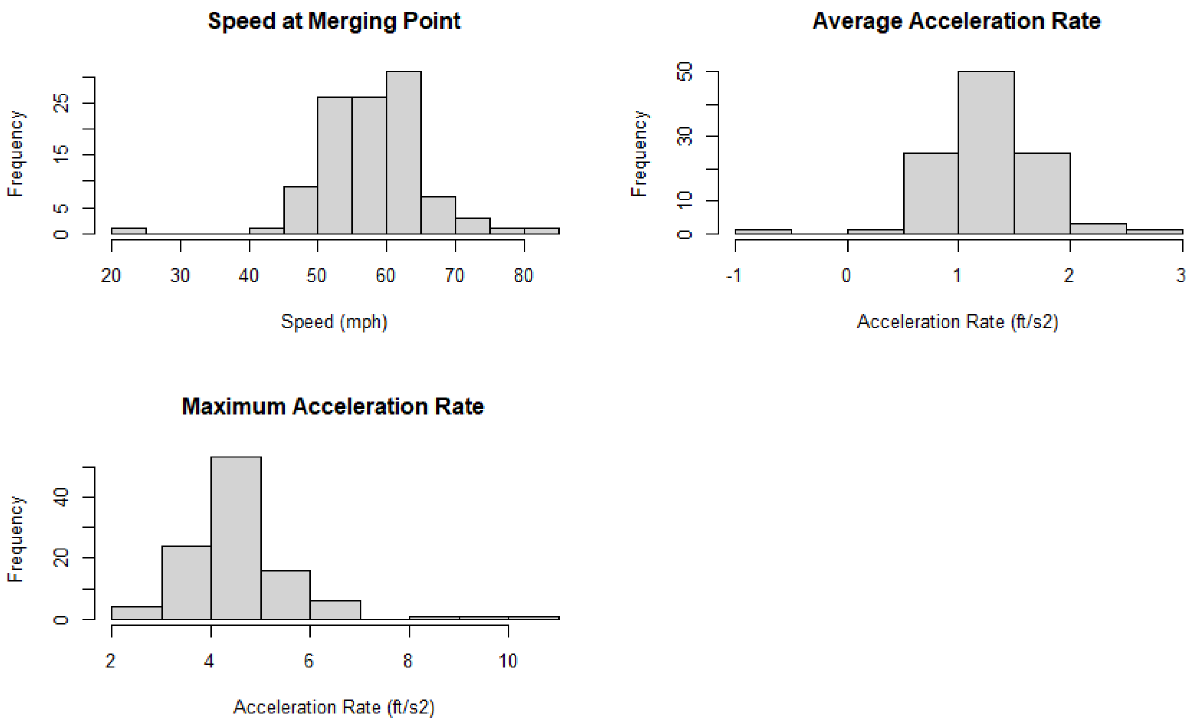


Figure 4-126. Distribution of Parameters for Site PA-5.

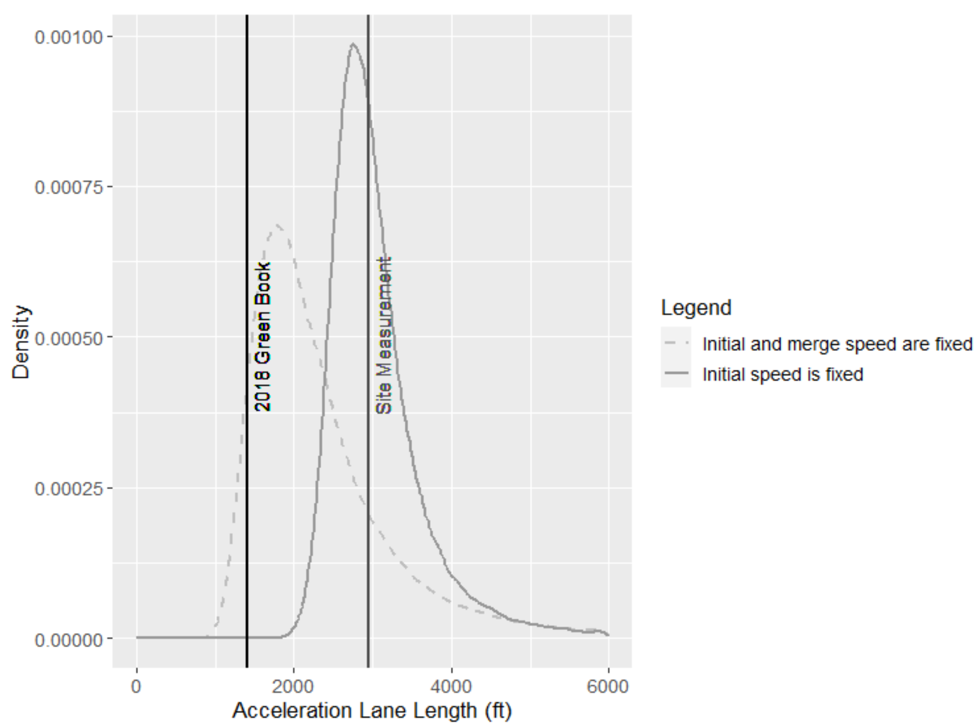


Figure 4-127. Frequency Distribution of Acceleration Lane Length based on Average Acceleration Rate for Site PA-5.

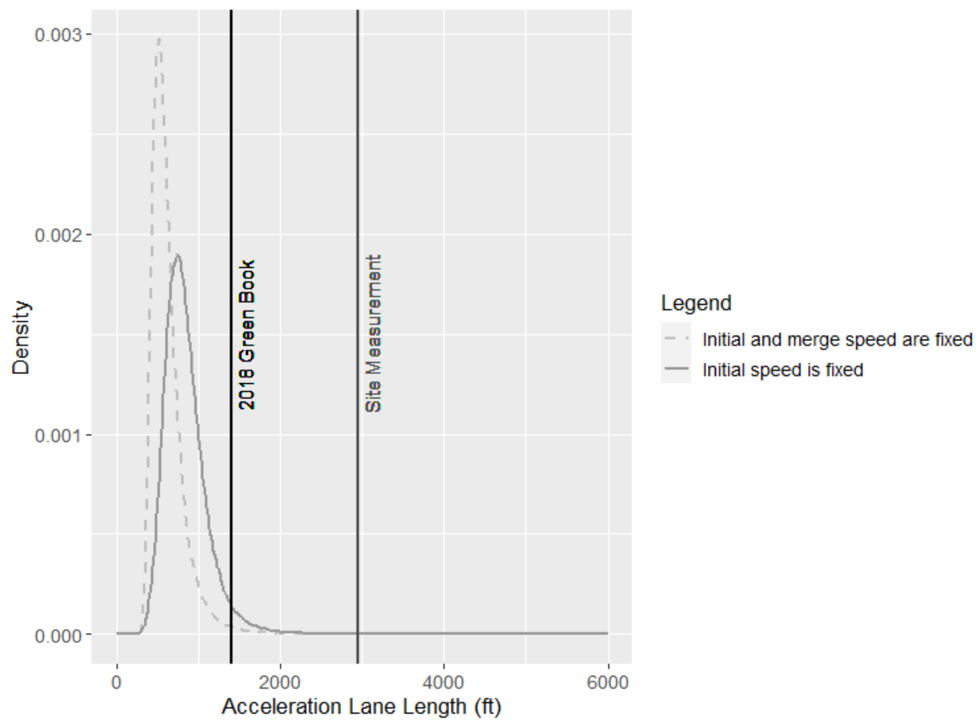


Figure 4-128. Frequency Distribution of Acceleration Lane Length based on Maximum Acceleration Rate for Site PA-5.

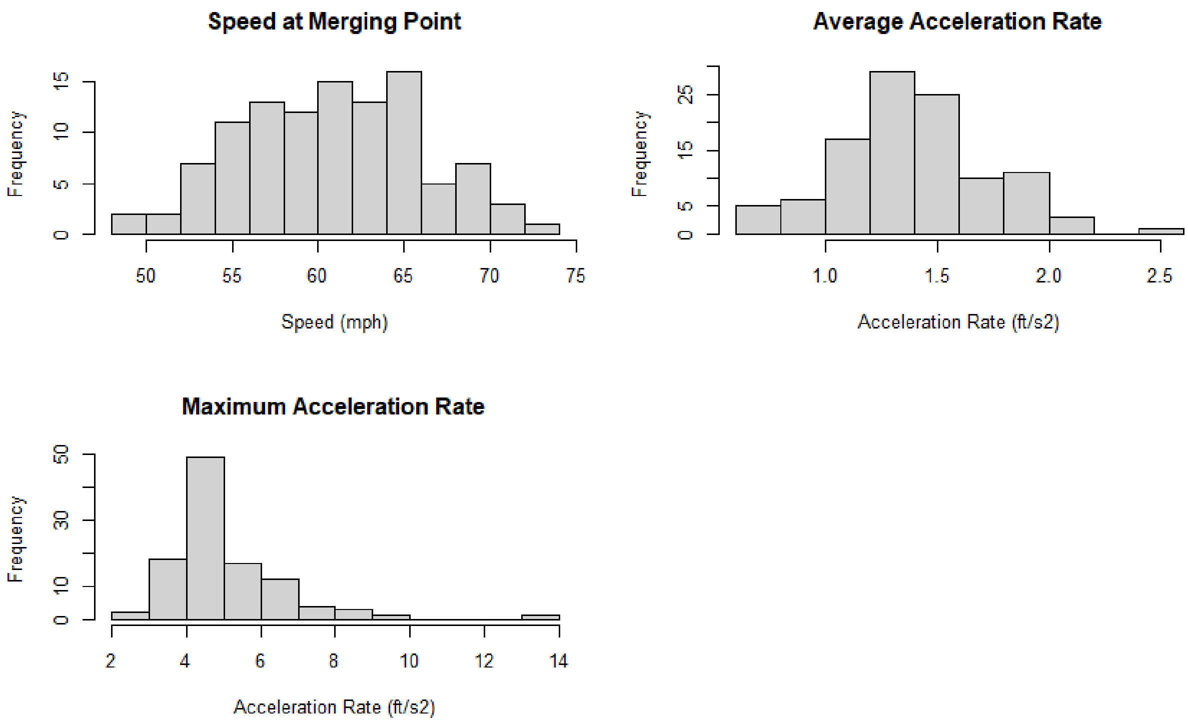


Figure 4-129. Distribution of Parameters for Site PA-6.

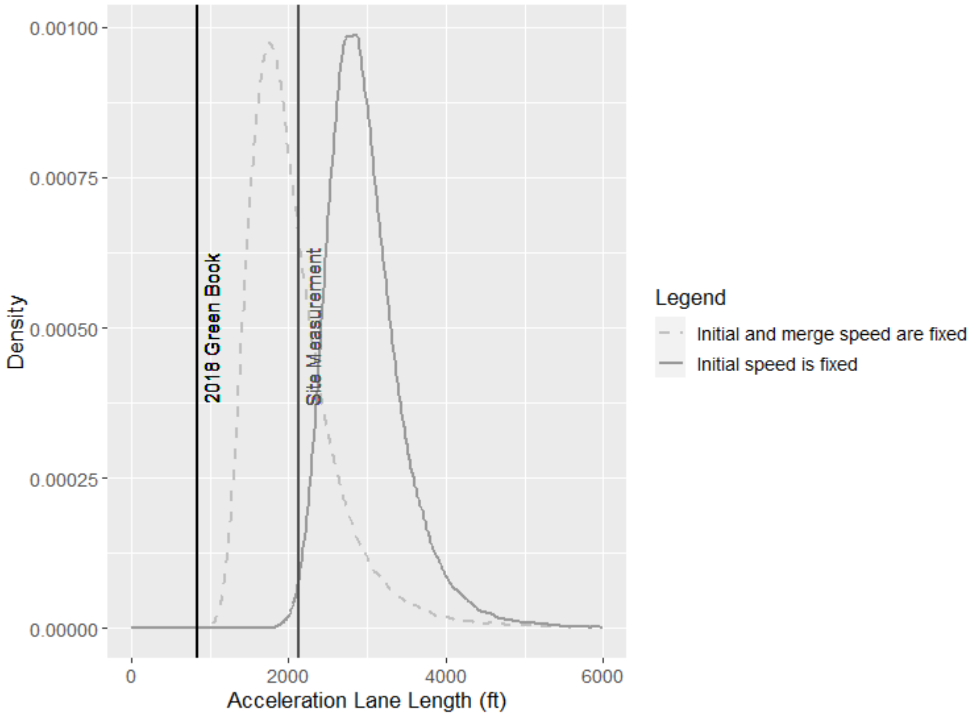


Figure 4-130. Frequency Distribution of Acceleration Lane Length based on Average Acceleration Rate for Site PA-6.

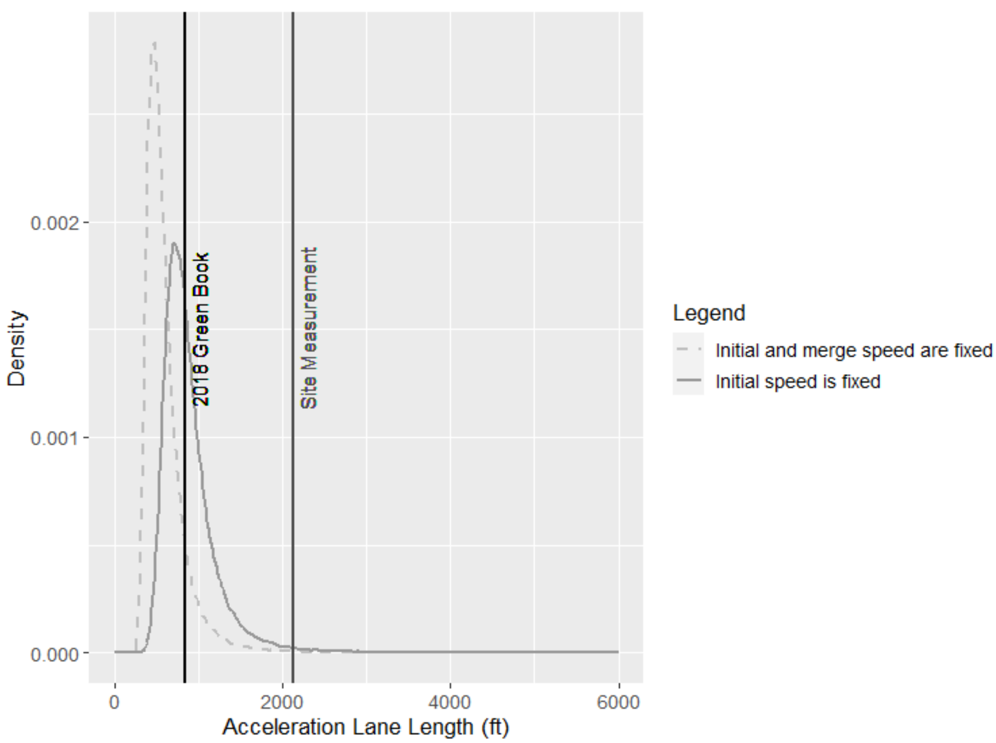


Figure 4-131. Frequency Distribution of Acceleration Lane Length based on Maximum Acceleration Rate for Site PA-6.

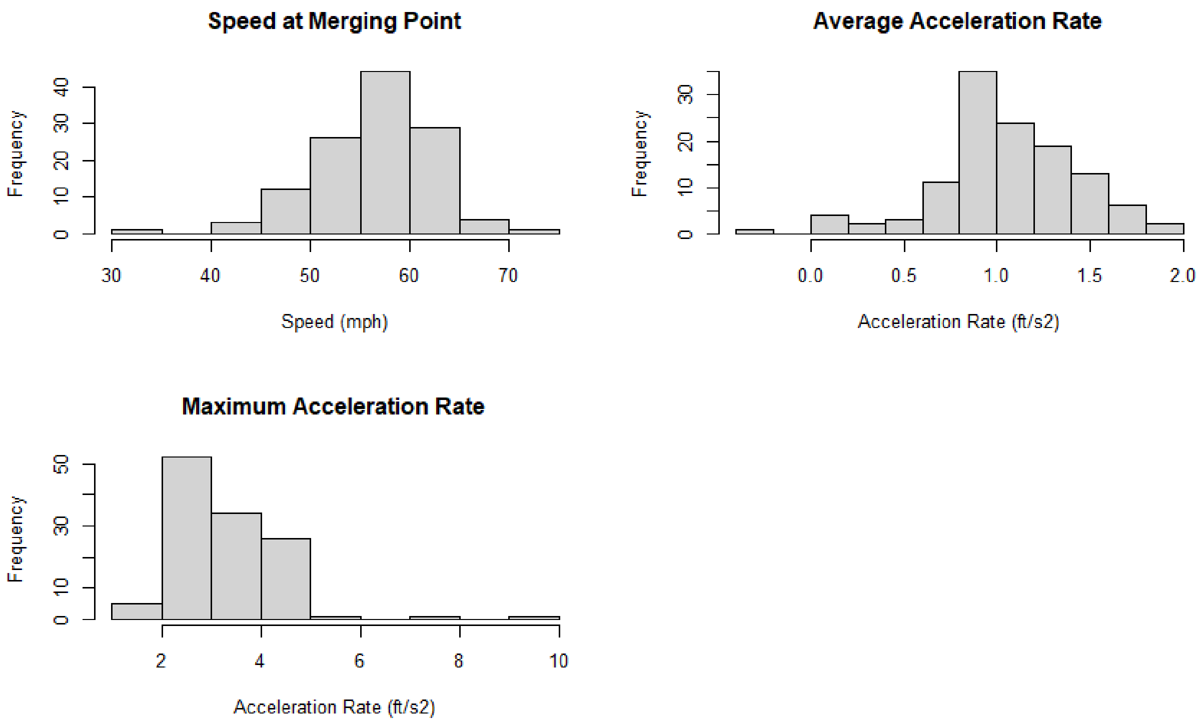


Figure 4-132. Distribution of Parameters for Site PA-7.

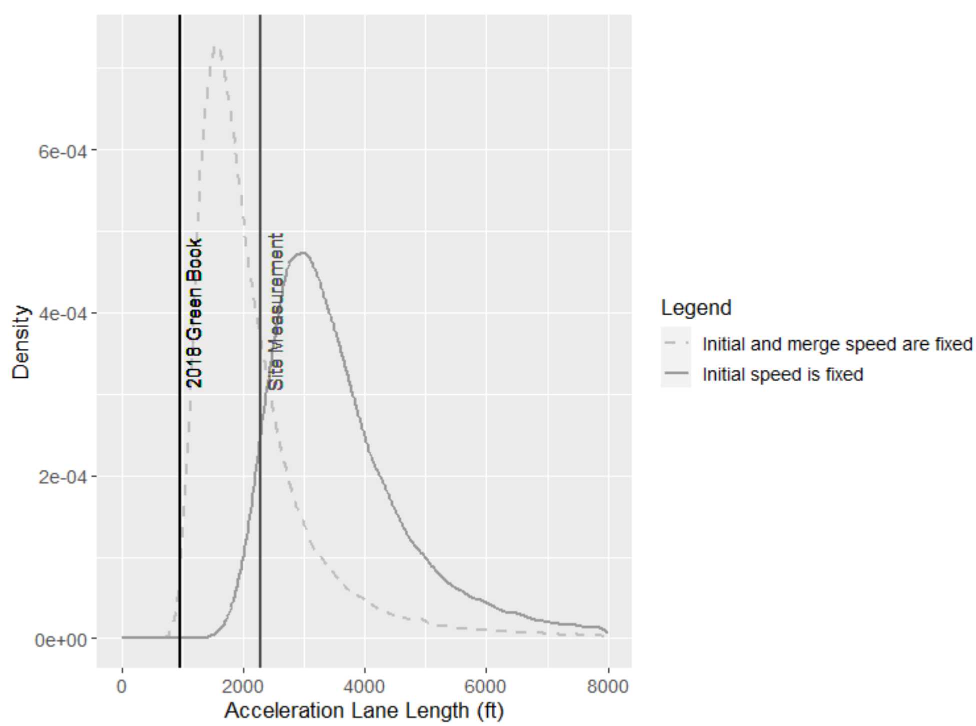


Figure 4-133. Frequency Distribution of Acceleration Lane Length based on Average Acceleration Rate for Site PA-7.

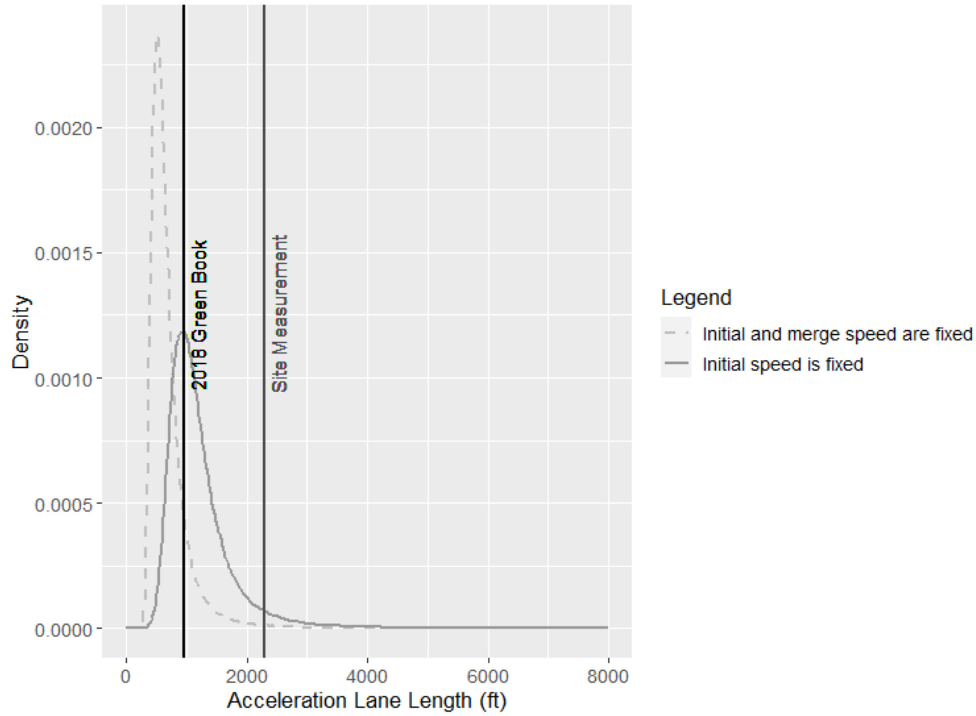


Figure 4-134. Frequency Distribution of Acceleration Lane Length based on Maximum Acceleration Rate for Site PA-7.

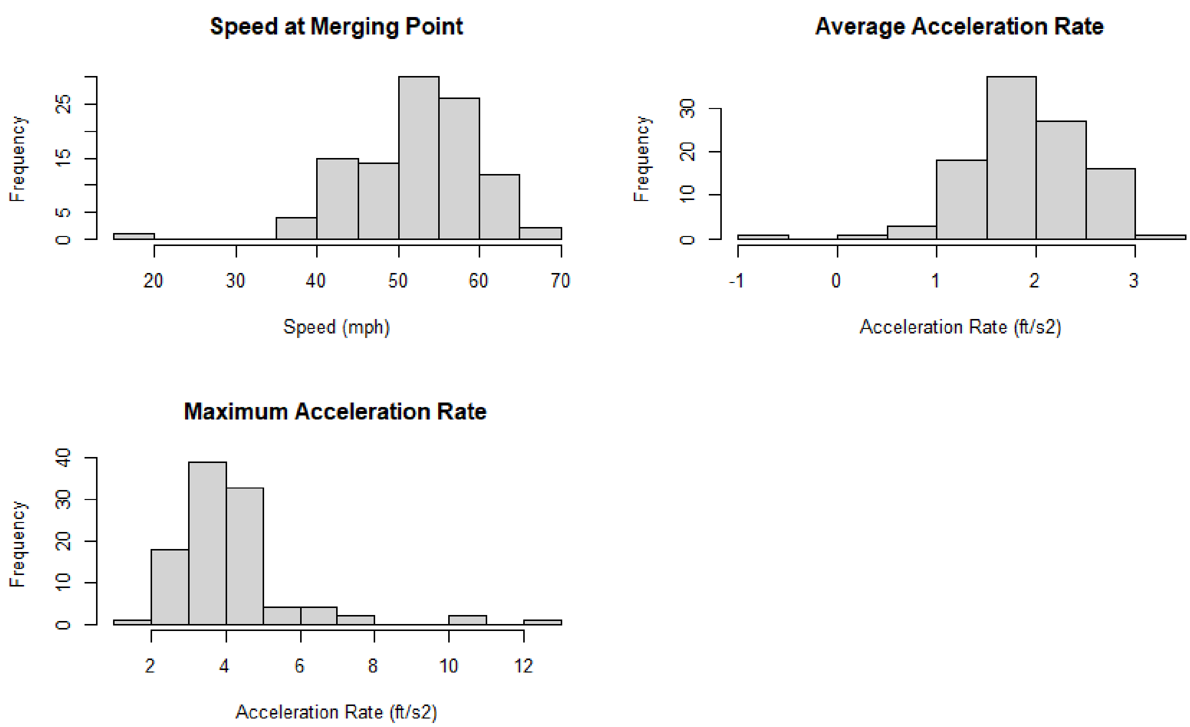


Figure 4-135. Distribution of Parameters for Site PA-8.

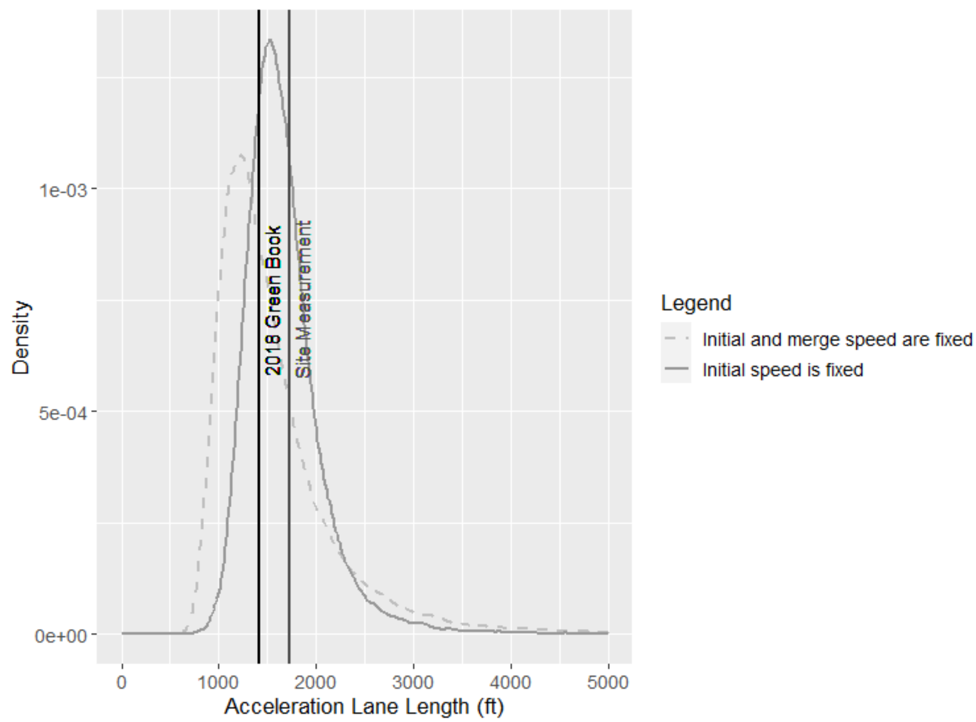


Figure 4-136. Frequency Distribution of Acceleration Lane Length based on Average Acceleration Rate for Site PA-8.

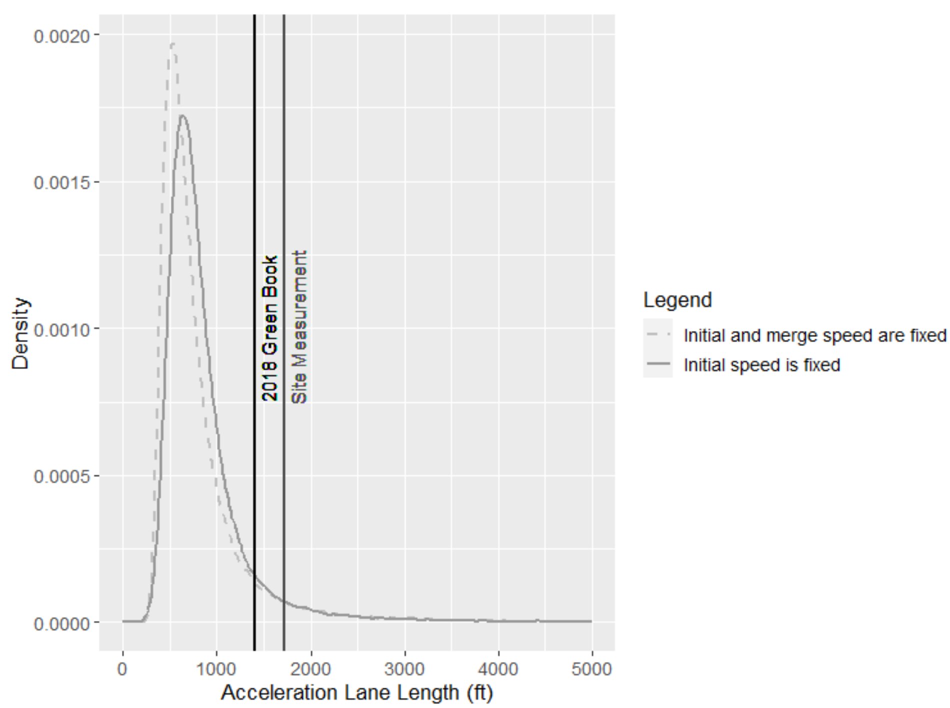


Figure 4-137. Frequency Distribution of Acceleration Lane Length based on Maximum Acceleration Rate for Site PA-8.

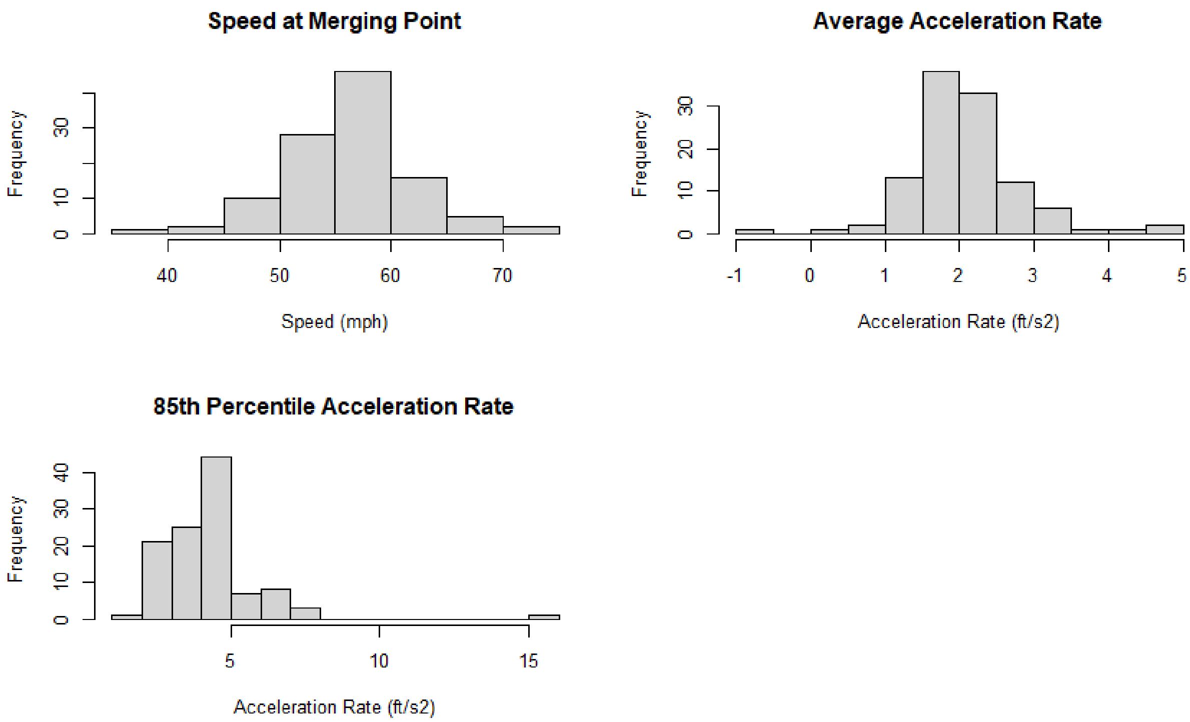


Figure 4-138. Distribution of Parameters for Site PA-9.

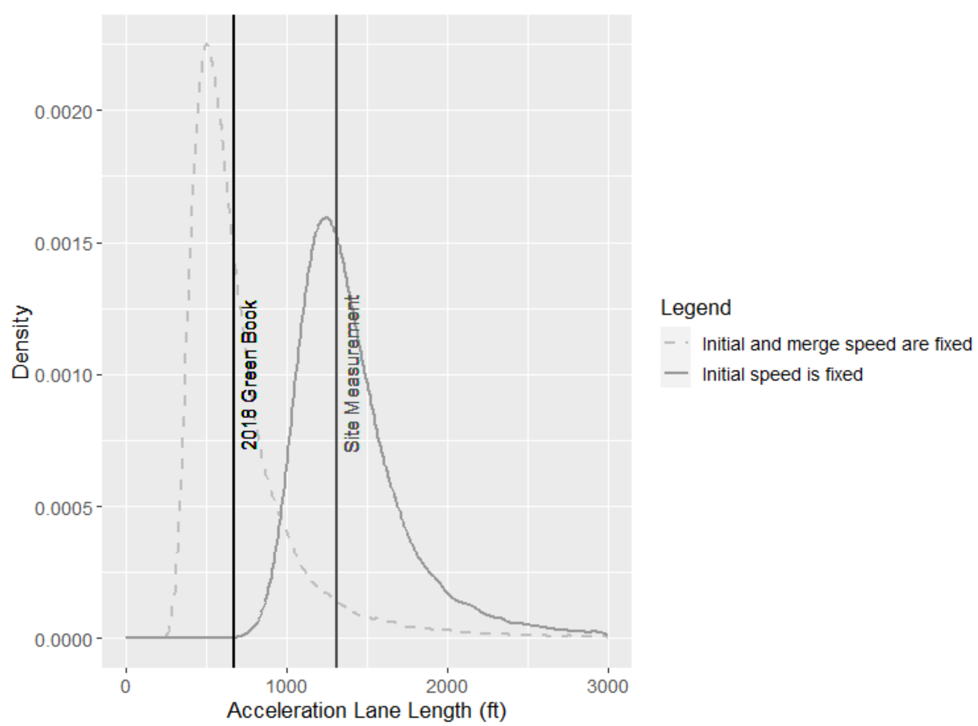


Figure 4-139. Frequency Distribution of Acceleration Lane Length based on Average Acceleration Rate for Site PA-9.

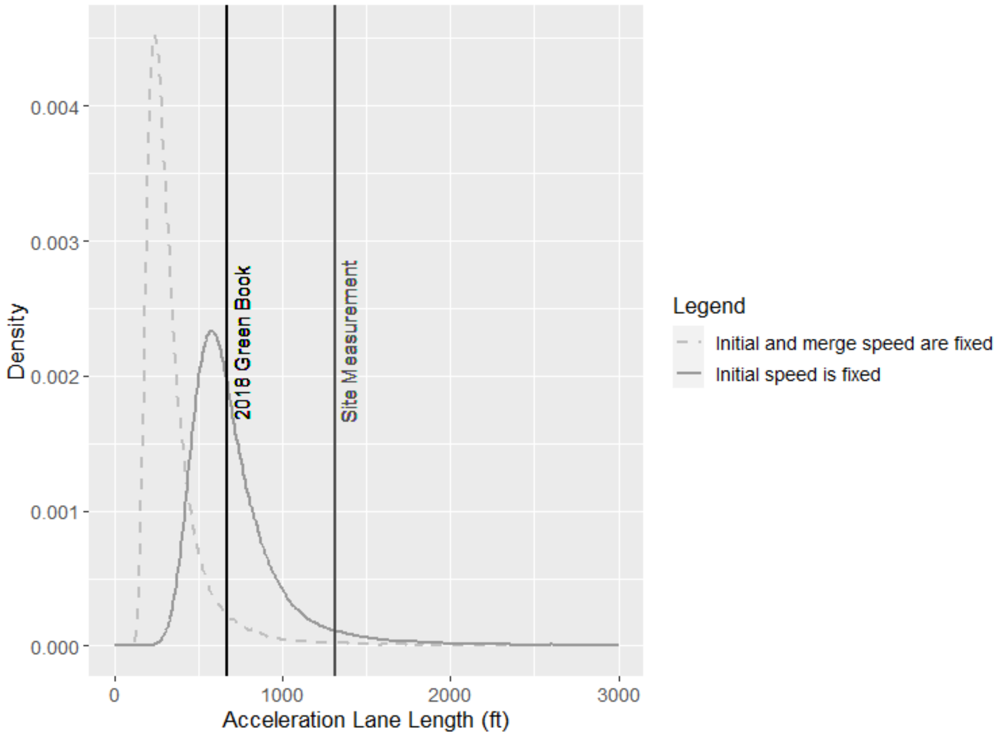


Figure 4-140. Frequency Distribution of Acceleration Lane Length based on Maximum Acceleration Rate for Site PA-9.

Appendix C Reliability-Based Analysis for Exit Ramps

This section presents the frequency distributions of speed and deceleration parameters utilized in the reliability-based analyses for exit ramps. The frequency distribution of demand deceleration lane lengths based on maximum and average deceleration rates, and its comparison with the field measured deceleration lane length and AASHTO Green Book-recommended values are also presented separately for each site.

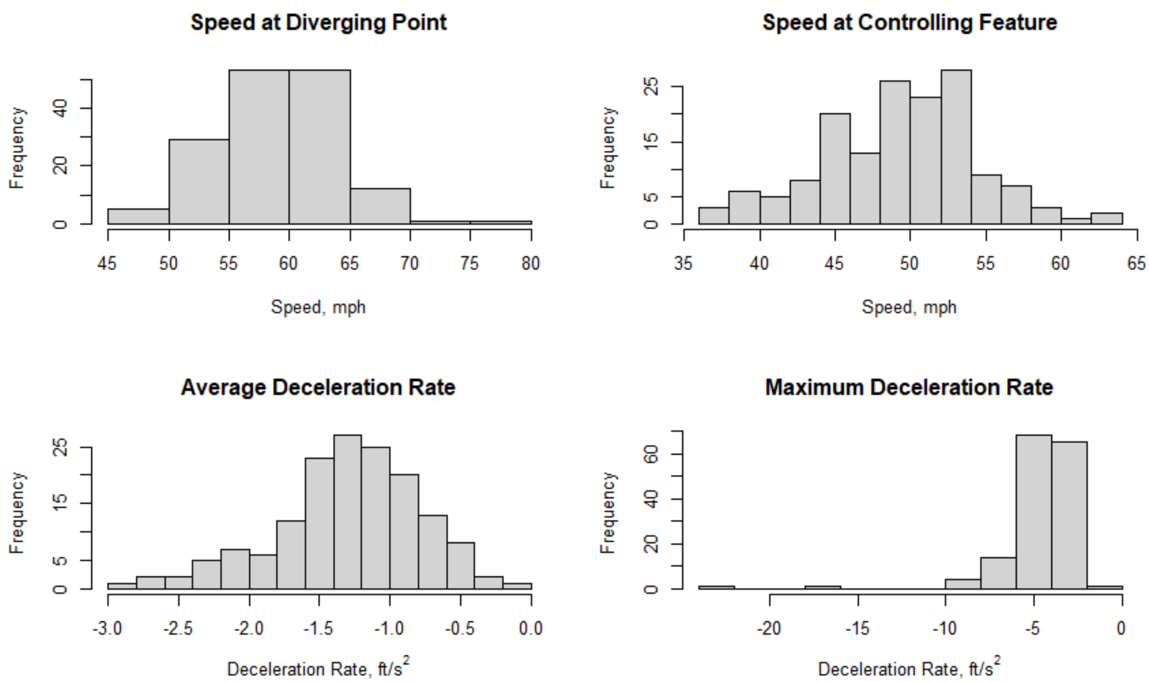


Figure 4-141. Distribution of Parameters for Site CA-11.

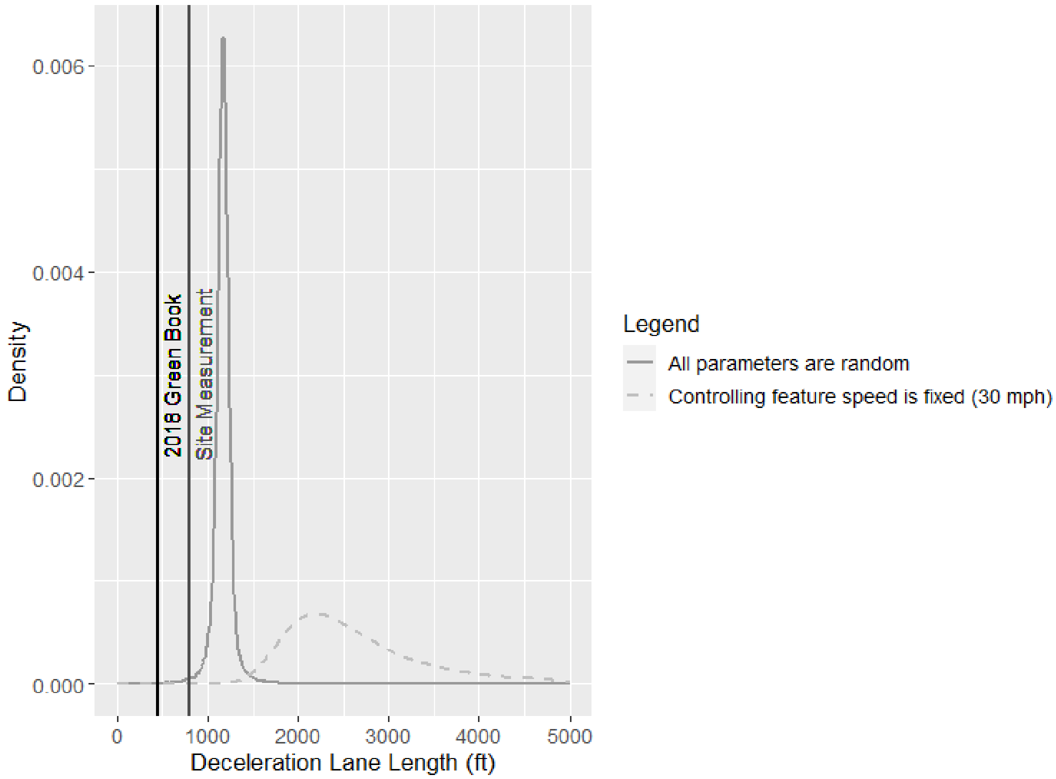


Figure 4-142. Frequency Distribution of Deceleration Lane Length based on Average Deceleration Rate for Site CA-11.

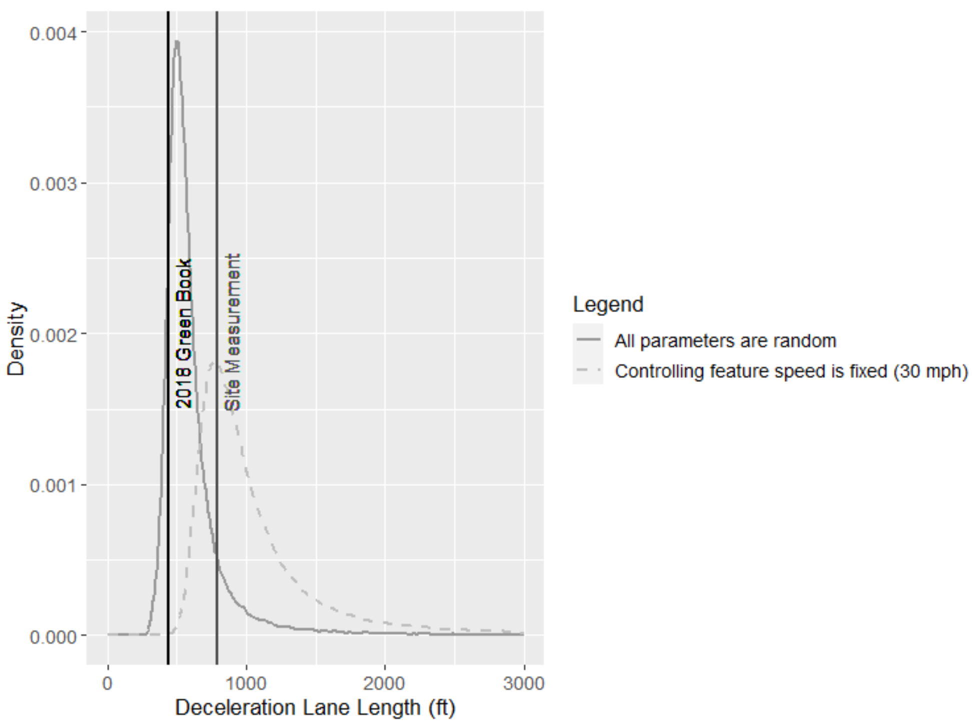


Figure 4-143. Frequency Distribution of Deceleration Lane Length based on Maximum Deceleration Rate for Site CA-11.

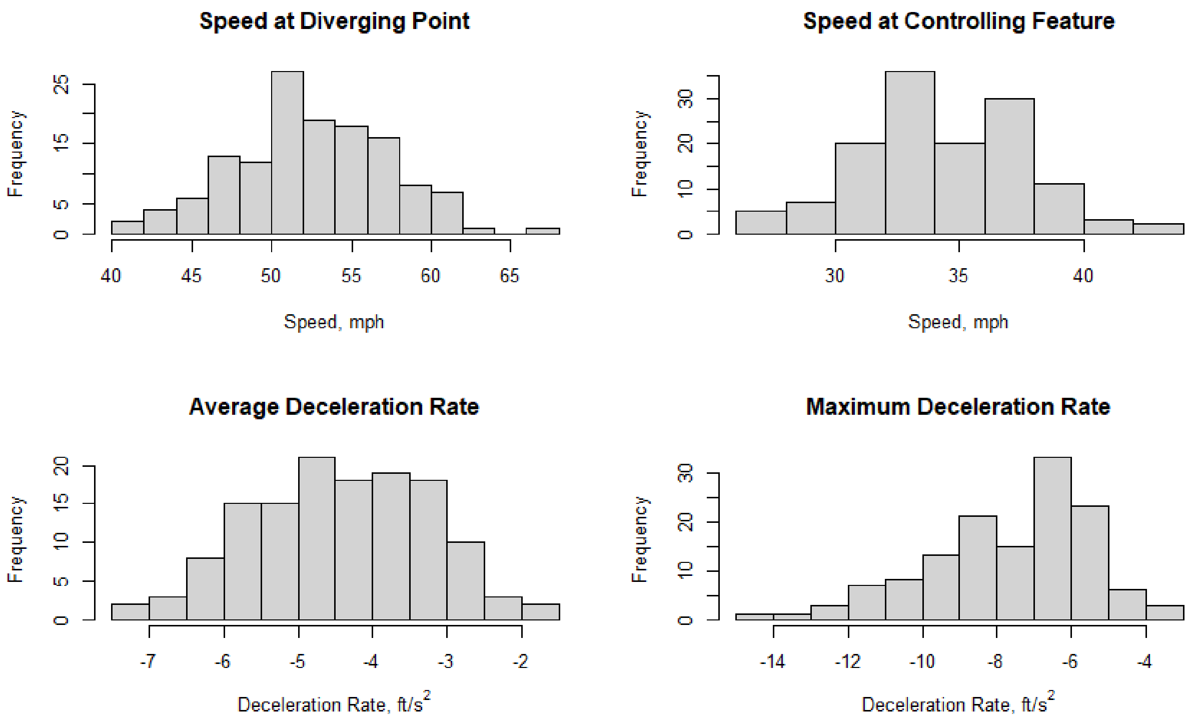


Figure 4-144. Distribution of Parameters for Site CA-12.

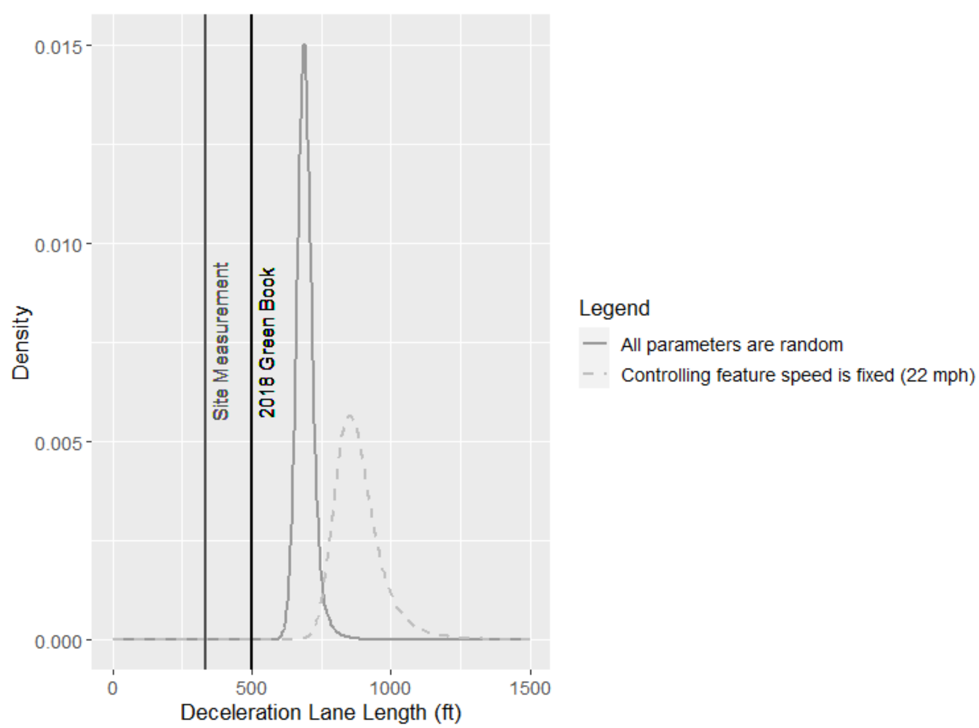


Figure 4-145. Frequency Distribution of Deceleration Lane Length based on Average Deceleration Rate for Site CA-12.

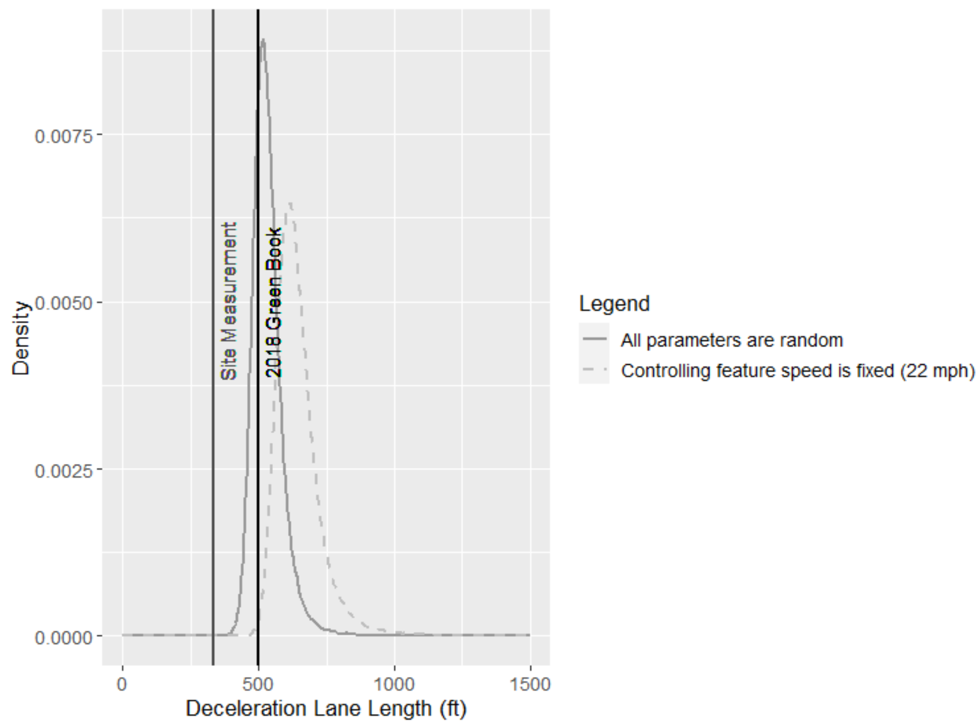


Figure 4-146. Frequency Distribution of Deceleration Lane Length based on Maximum Deceleration Rate for Site CA-12.

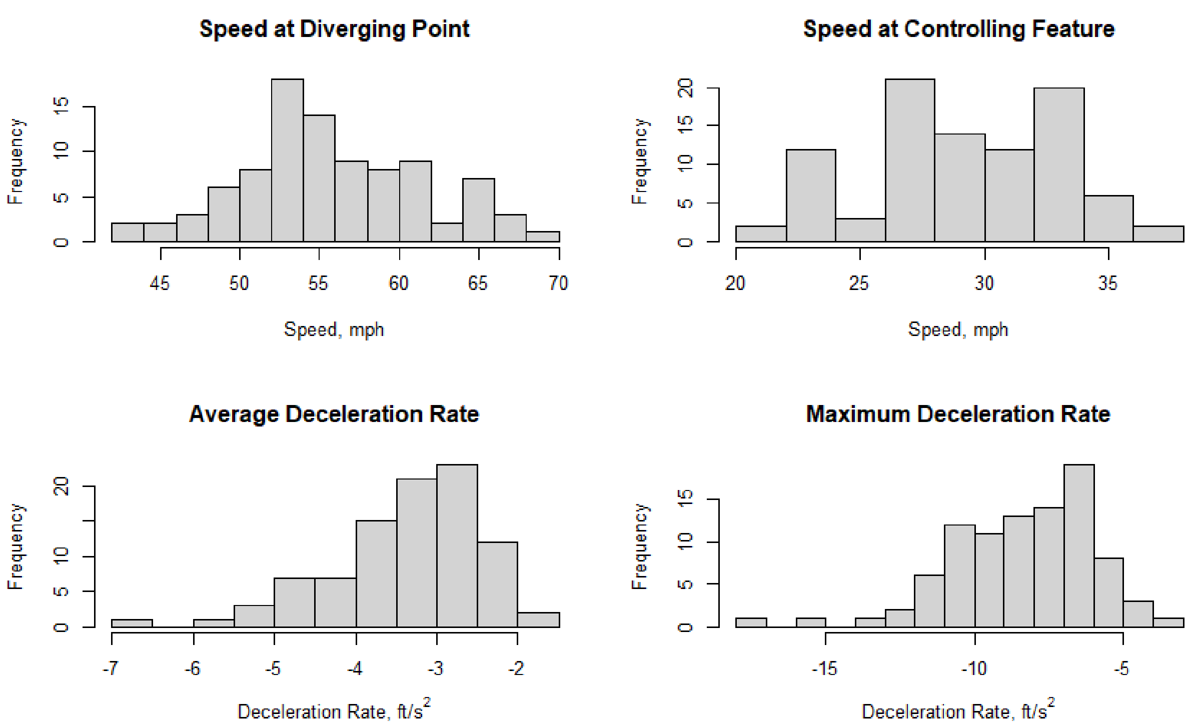


Figure 4-147. Distribution of Parameters for Site CA-13.

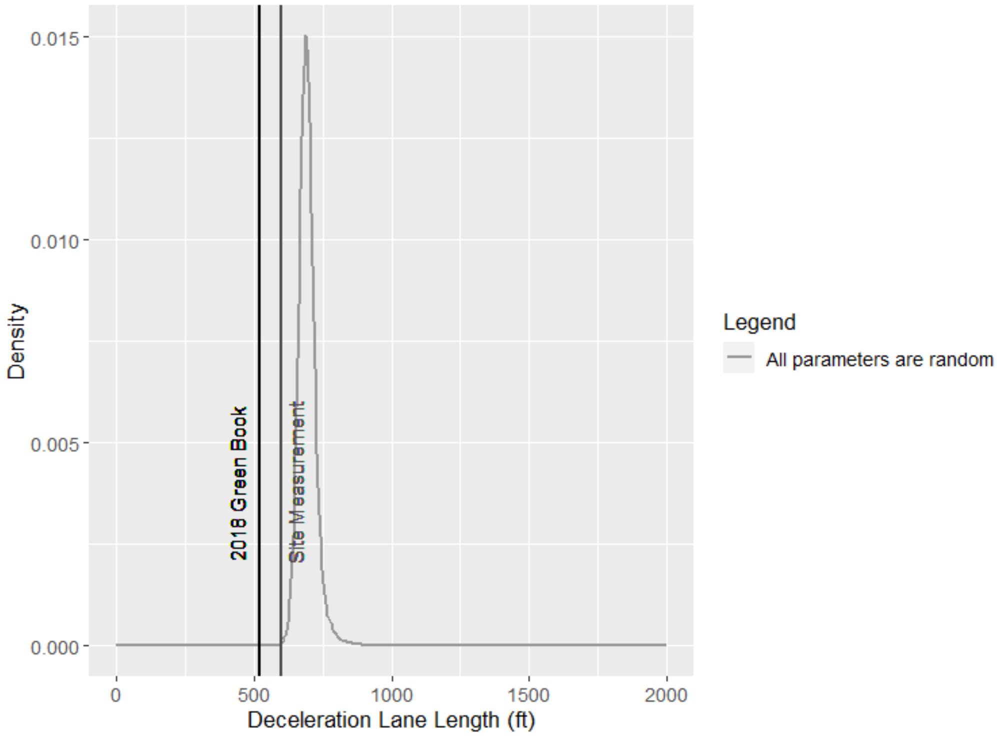


Figure 4-148. Frequency Distribution of Deceleration Lane Length based on Average Deceleration Rate for Site CA-13.

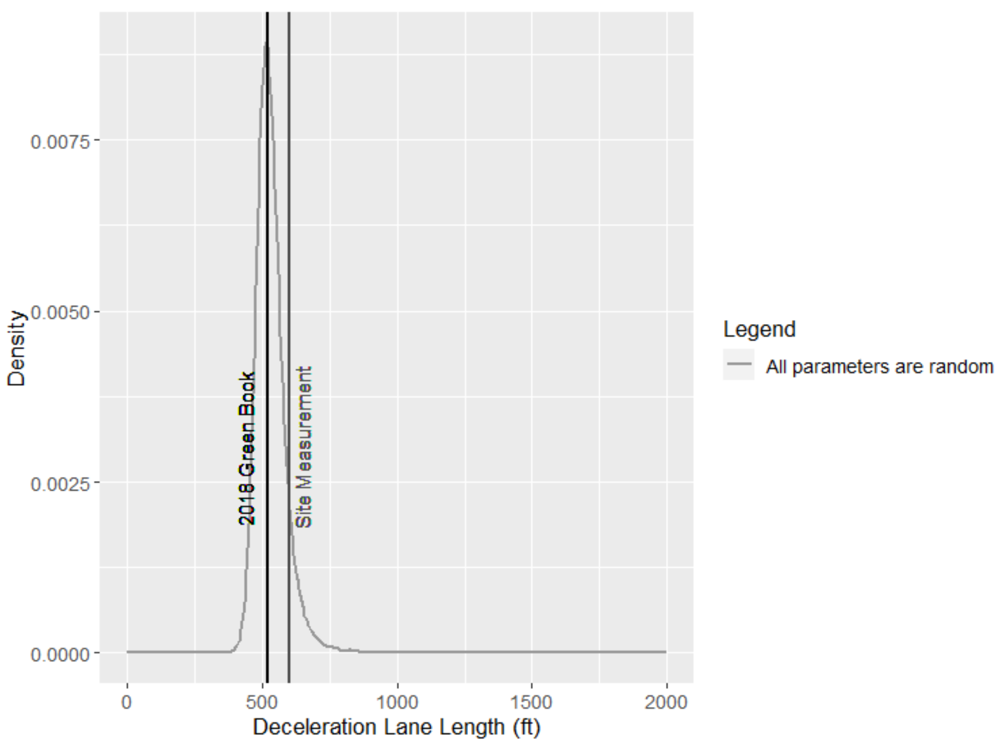


Figure 4-149. Frequency Distribution of Deceleration Lane Length based on Maximum Deceleration Rate for Site CA-13.

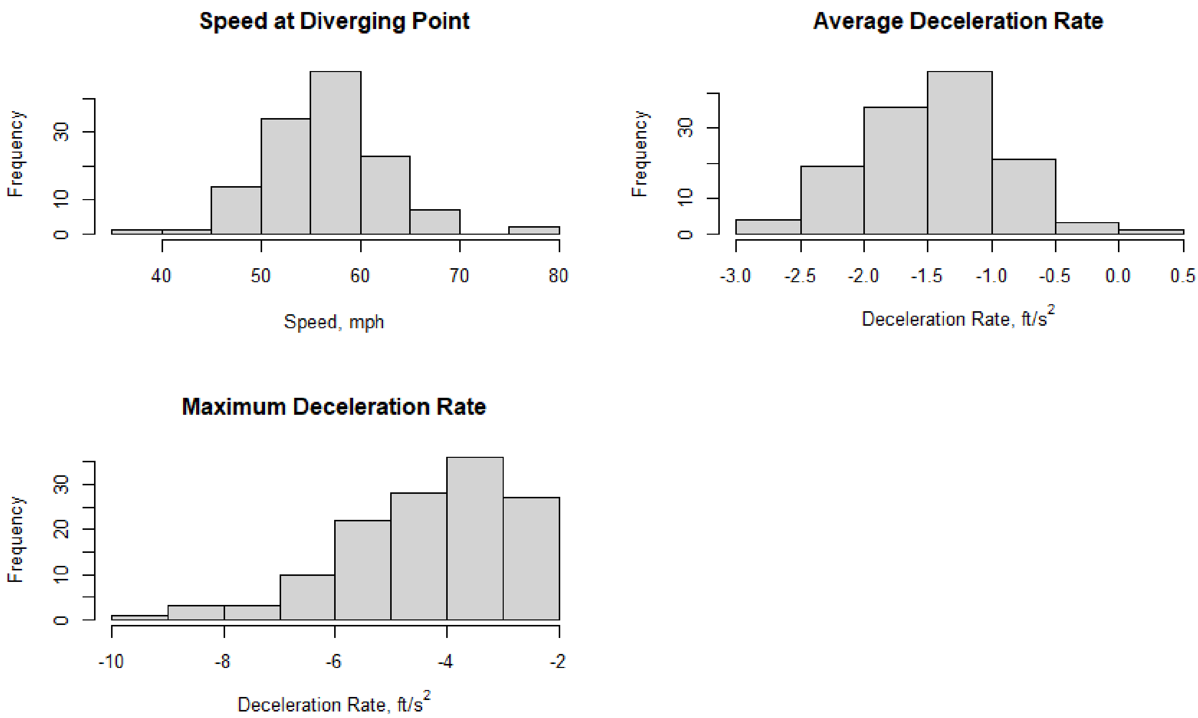


Figure 4-150. Distribution of Parameters for Site CA-14.

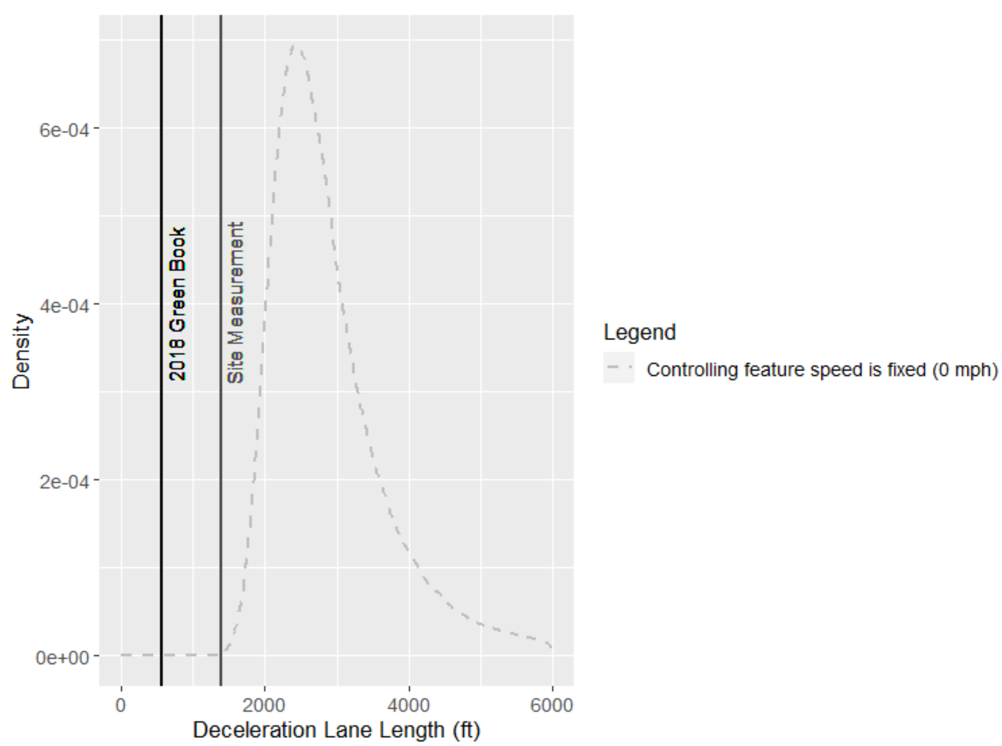


Figure 4-151. Frequency Distribution of Deceleration Lane Length based on Average Deceleration Rate for Site CA-14.

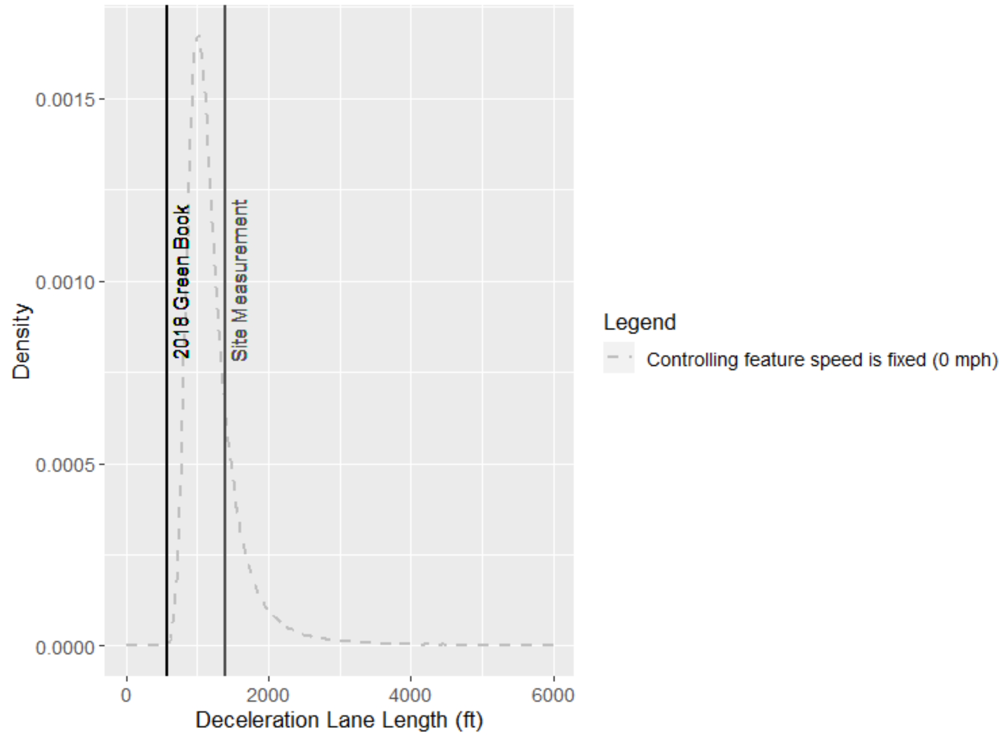


Figure 4-152. Frequency Distribution of Deceleration Lane Length based on Maximum Deceleration Rate for Site CA-14.

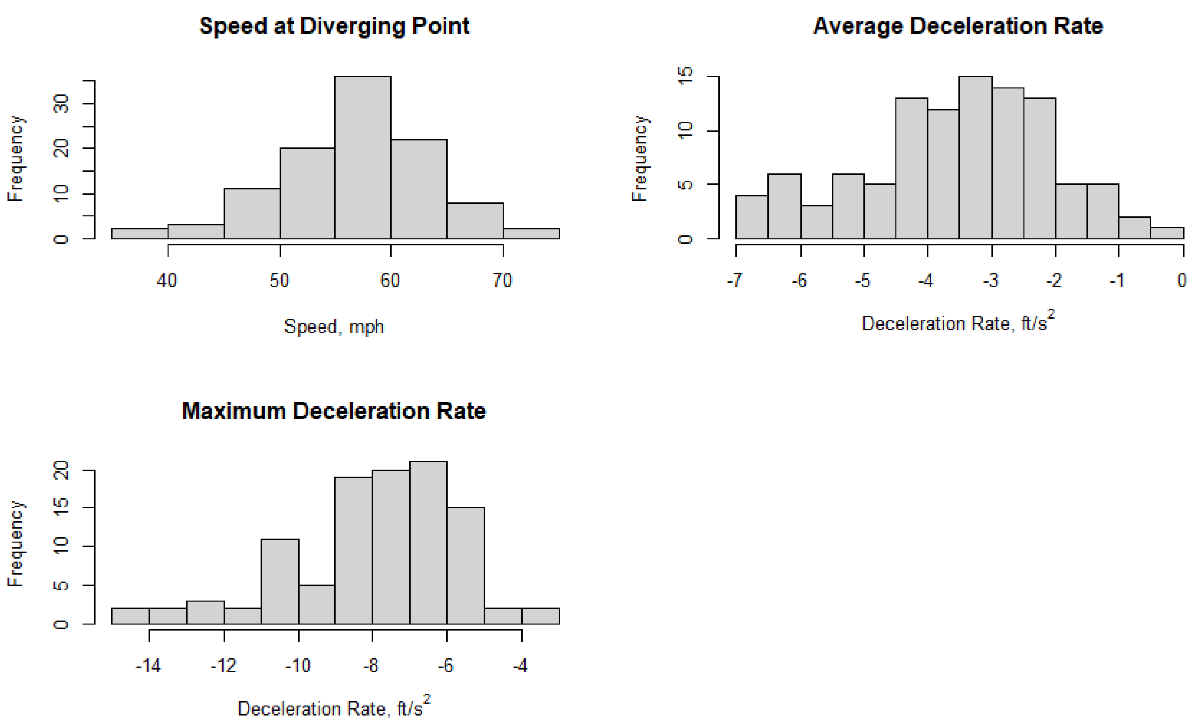


Figure 4-153. Distribution of Parameters for Site CA-15.

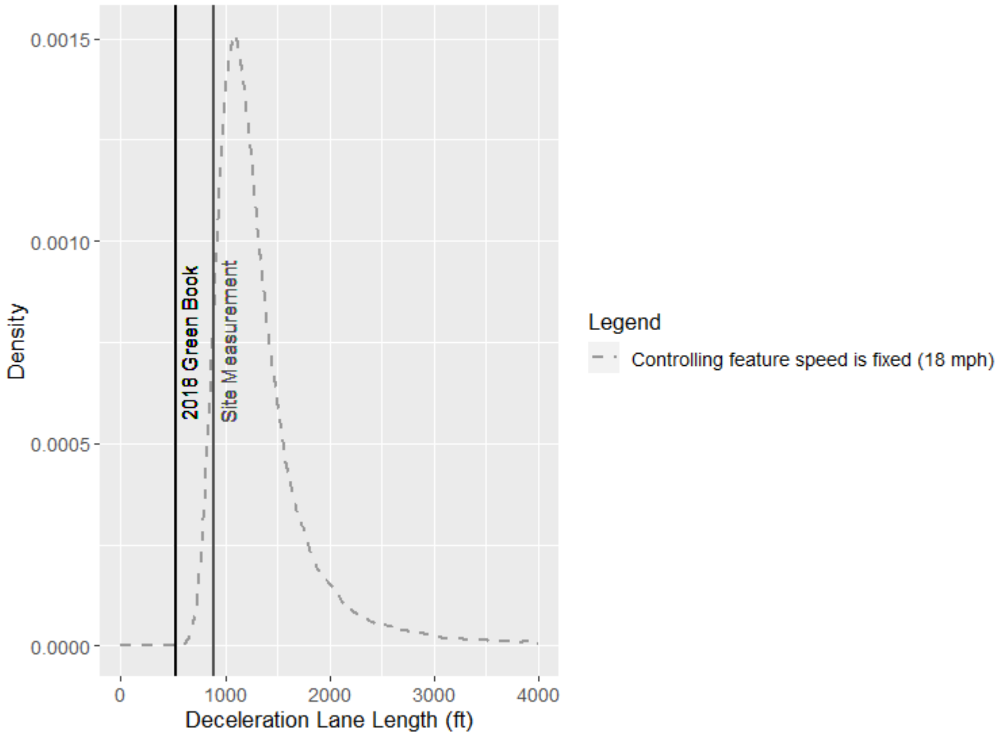


Figure 4-154. Frequency Distribution of Deceleration Lane Length based on Average Deceleration Rate for Site CA-15.

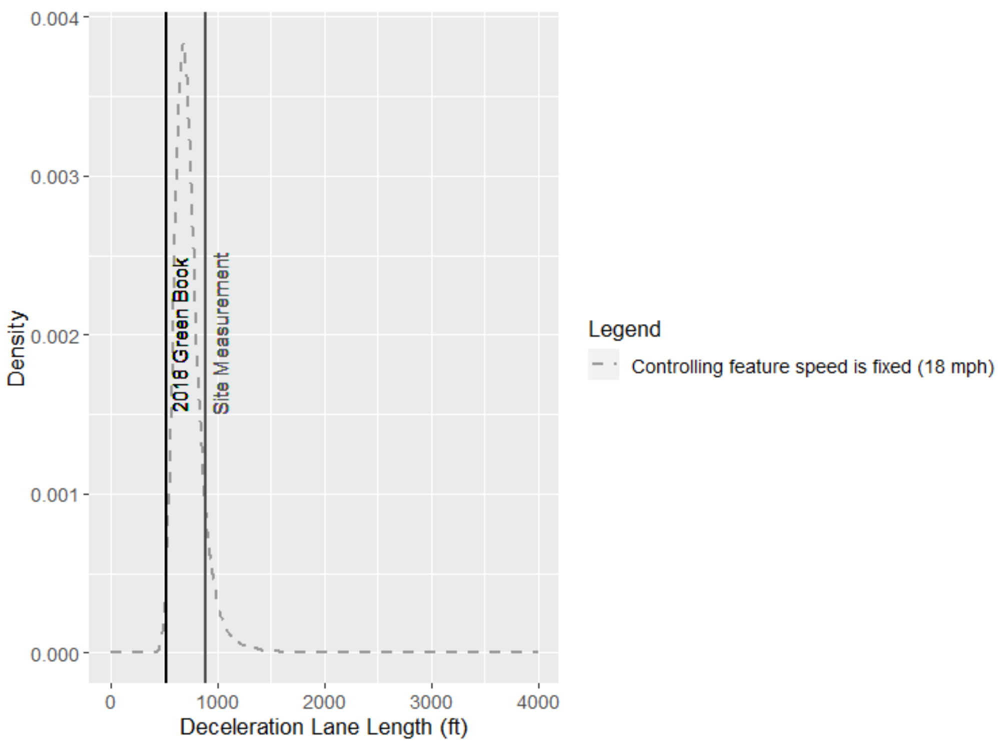


Figure 4-155. Frequency Distribution of Deceleration Lane Length based on Maximum Deceleration Rate for Site CA-15.

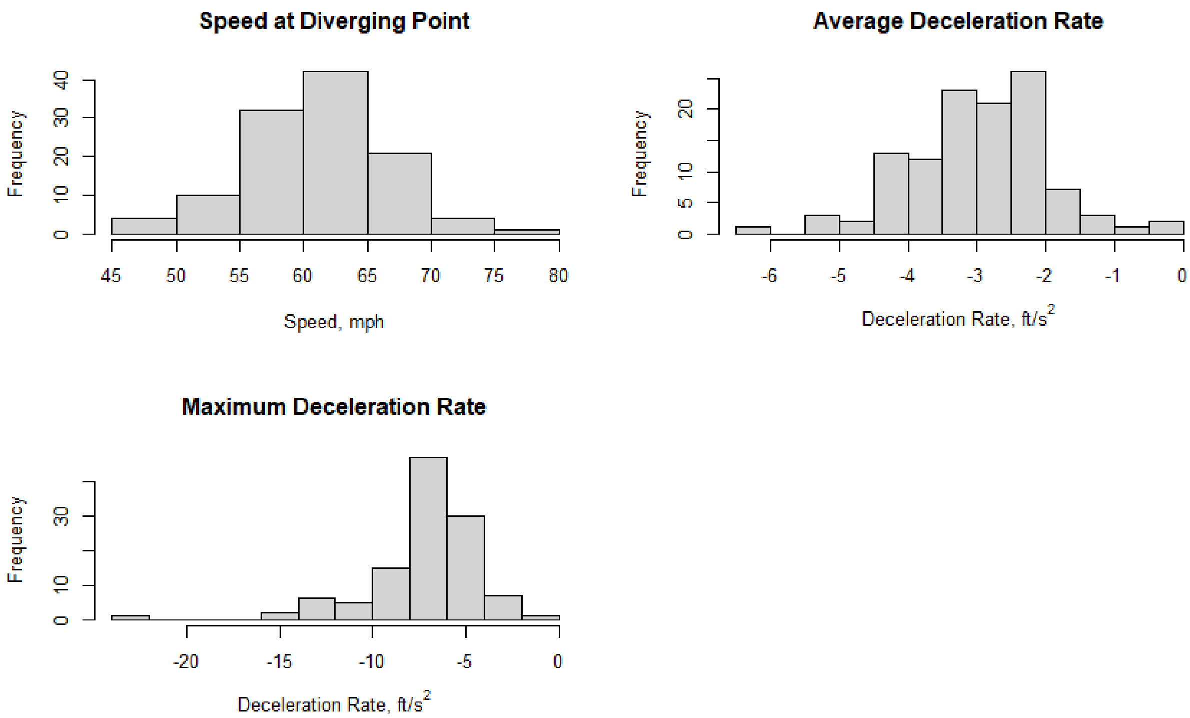


Figure 4-156. Distribution of Parameters for Site CA-16.

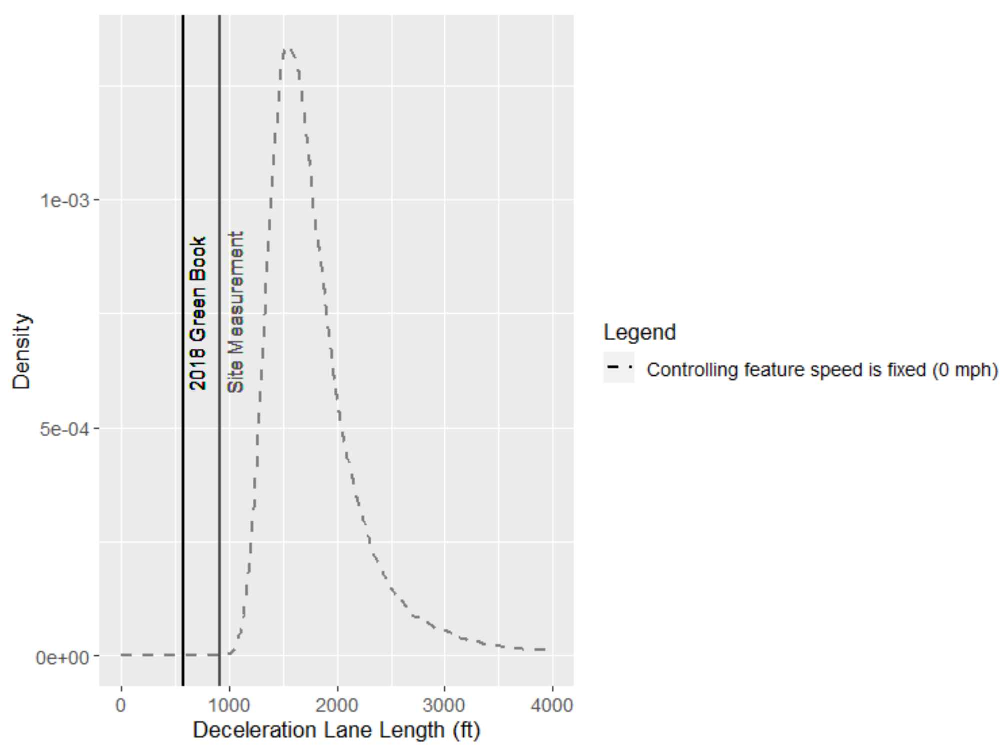


Figure 4-157. Frequency Distribution of Deceleration Lane Length based on Average Deceleration Rate for Site CA-16.

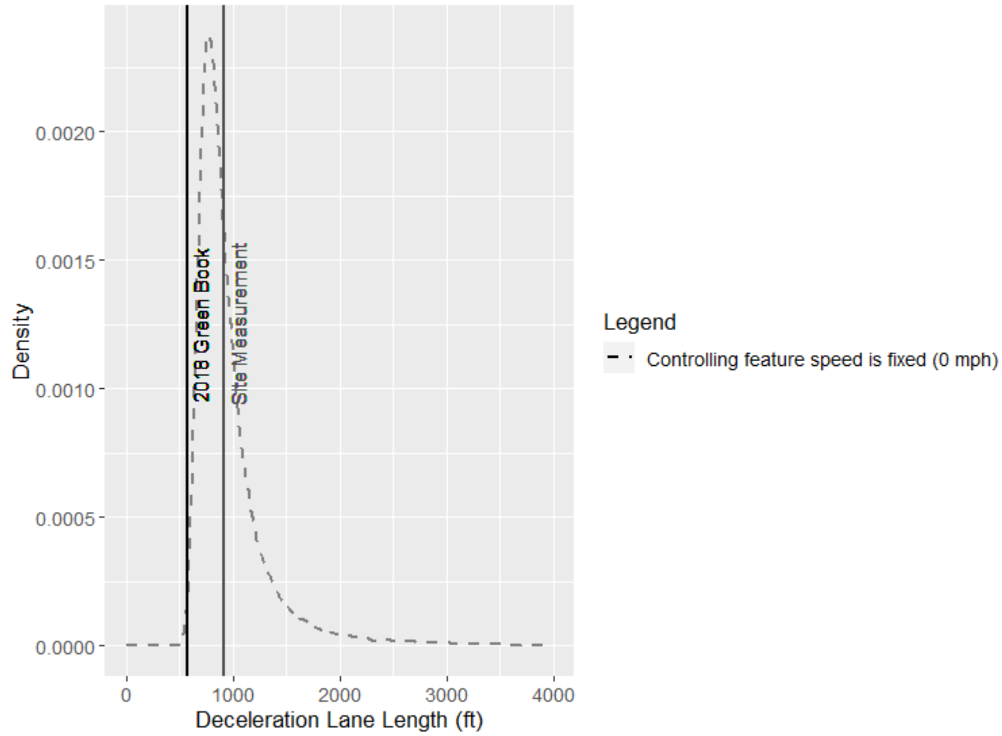


Figure 4-158. Frequency Distribution of Deceleration Lane Length based on Maximum Deceleration Rate for Site CA-16.

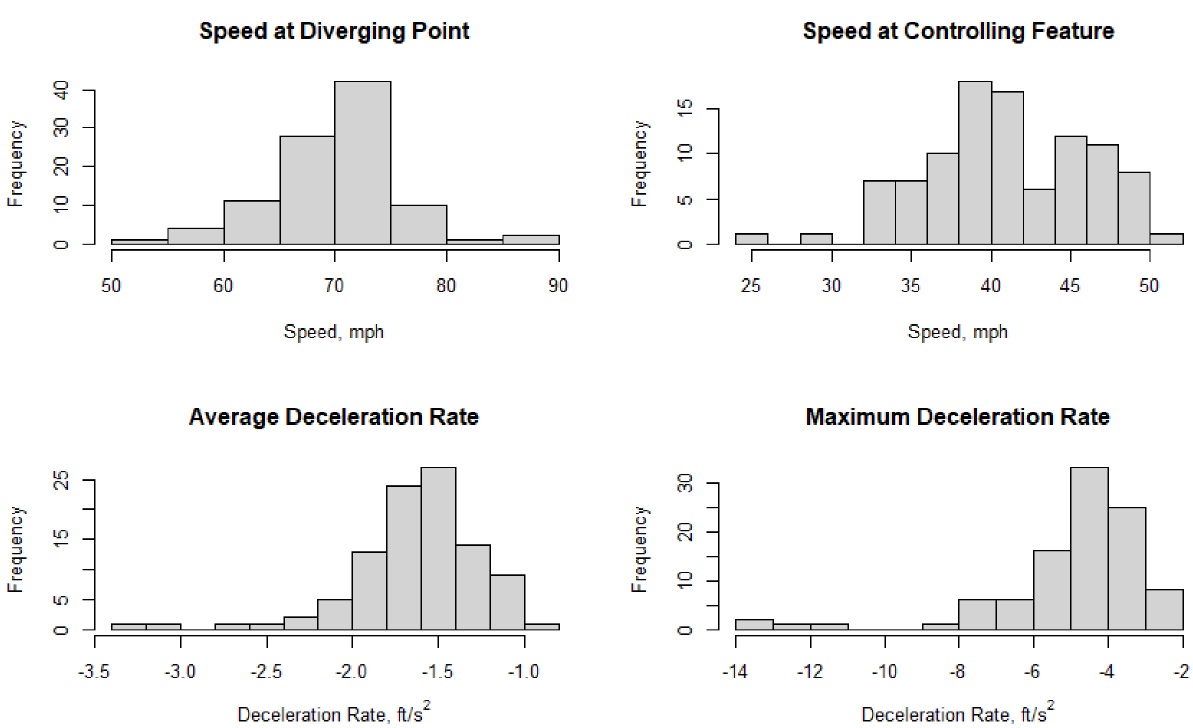


Figure 4-159. Distribution of Parameters for Site MI-9.

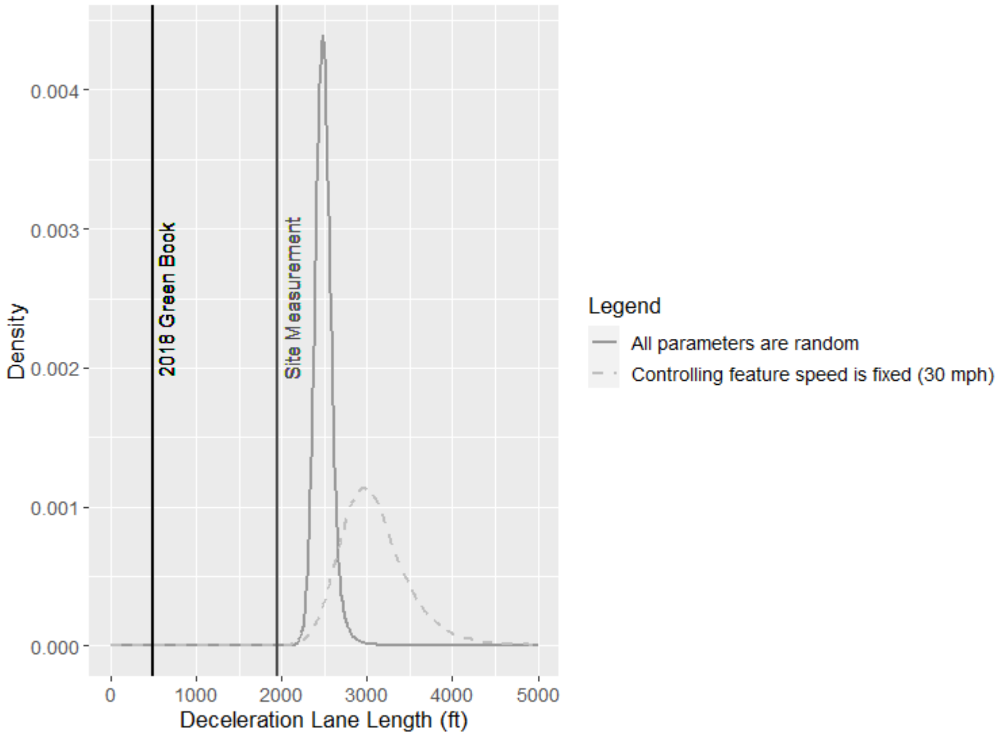


Figure 4-160. Frequency Distribution of Deceleration Lane Length based on Average Deceleration Rate for Site MI-9.

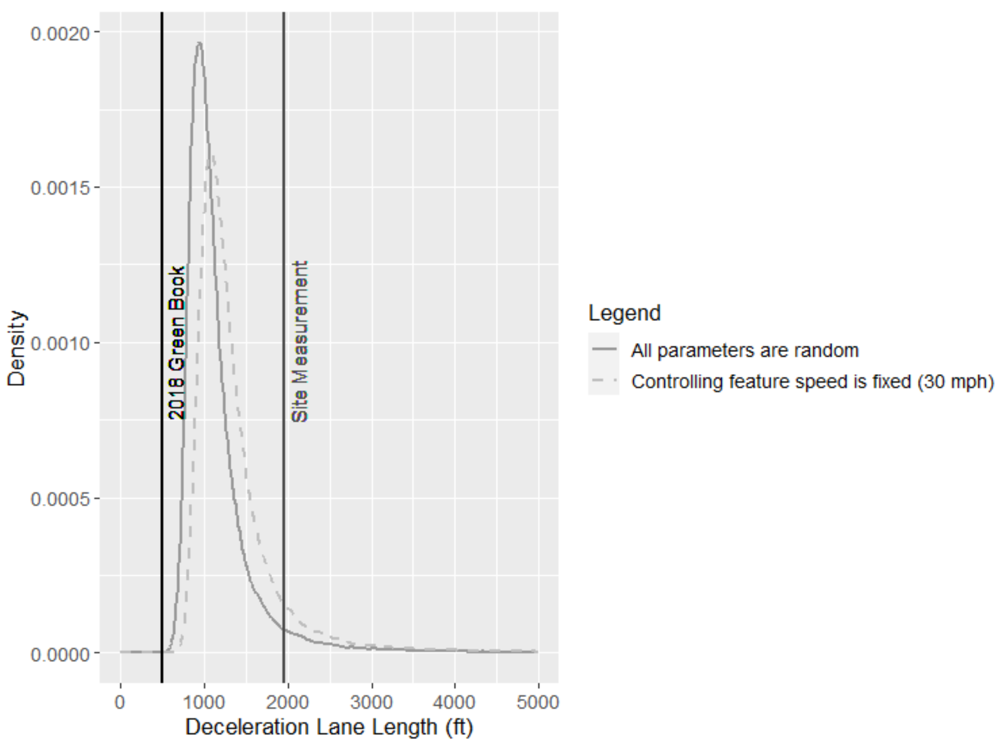


Figure 4-161. Frequency Distribution of Deceleration Lane Length based on Maximum Deceleration Rate for Site MI-9.

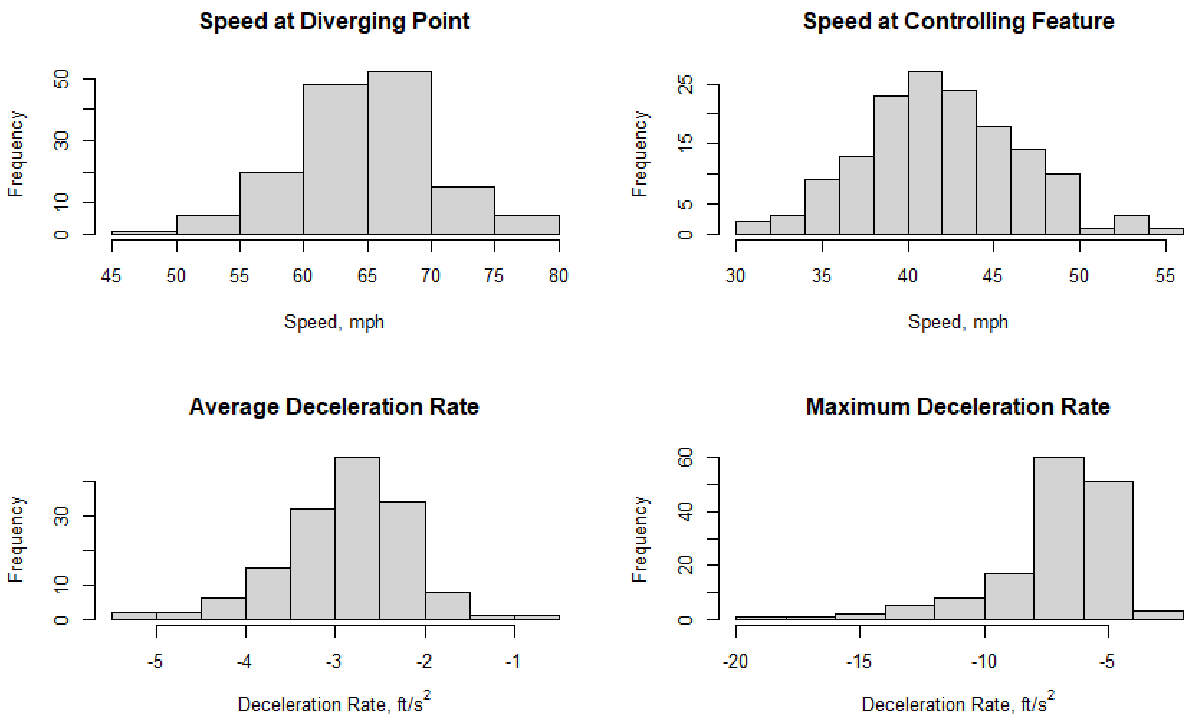


Figure 4-162 Distribution of Parameters for Site MI-10.

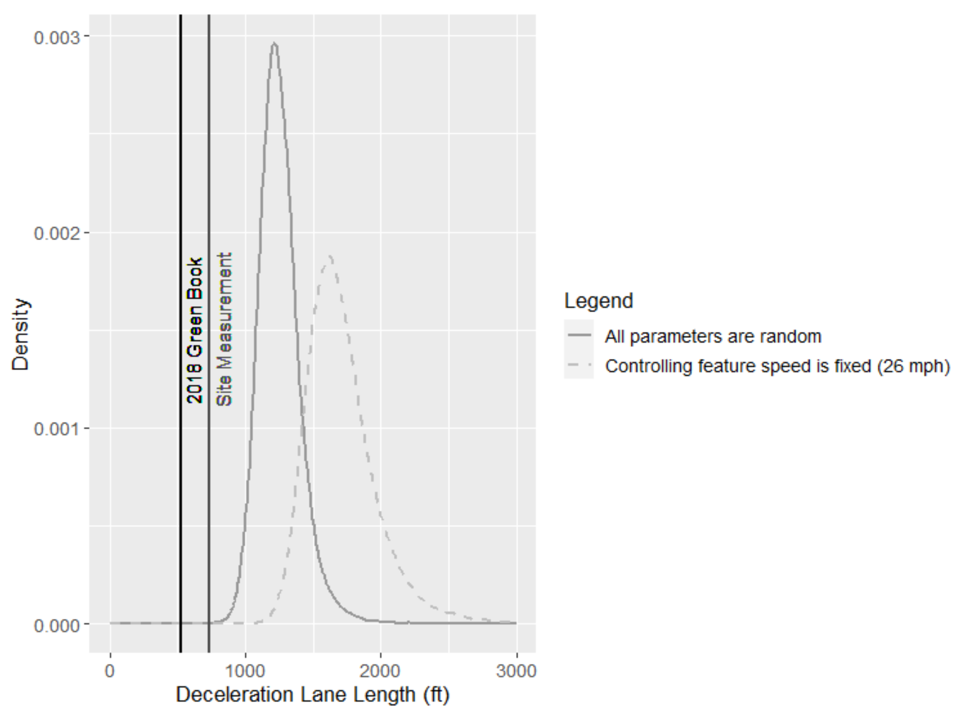


Figure 4-163. Frequency Distribution of Deceleration Lane Length based on Average Deceleration Rate for Site MI-10.

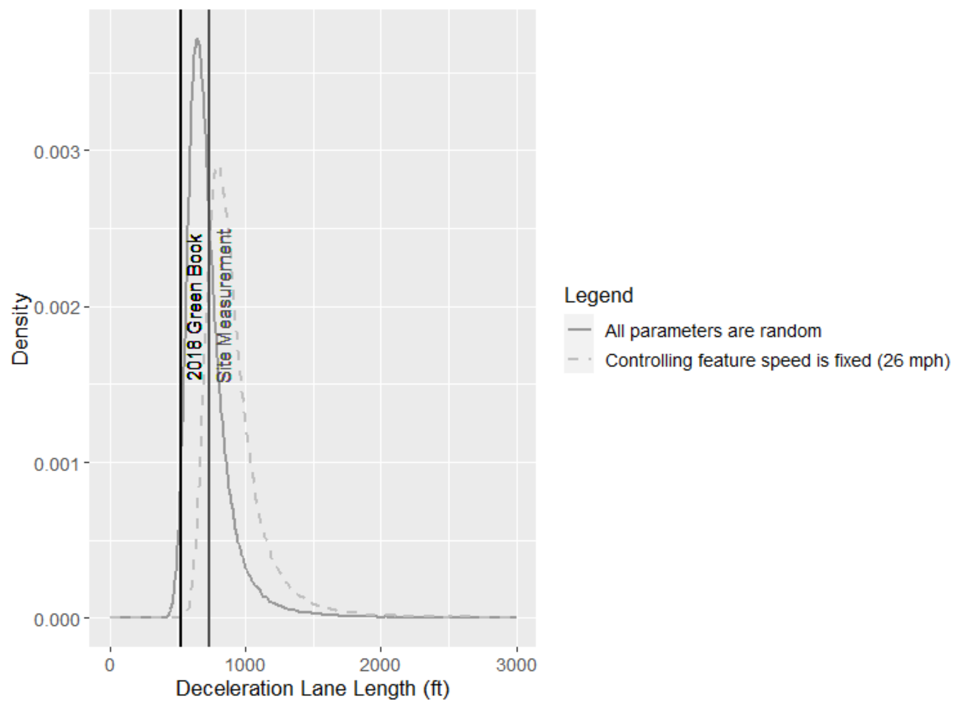


Figure 4-164. Frequency Distribution of Deceleration Lane Length based on Maximum Deceleration Rate for Site MI-10.

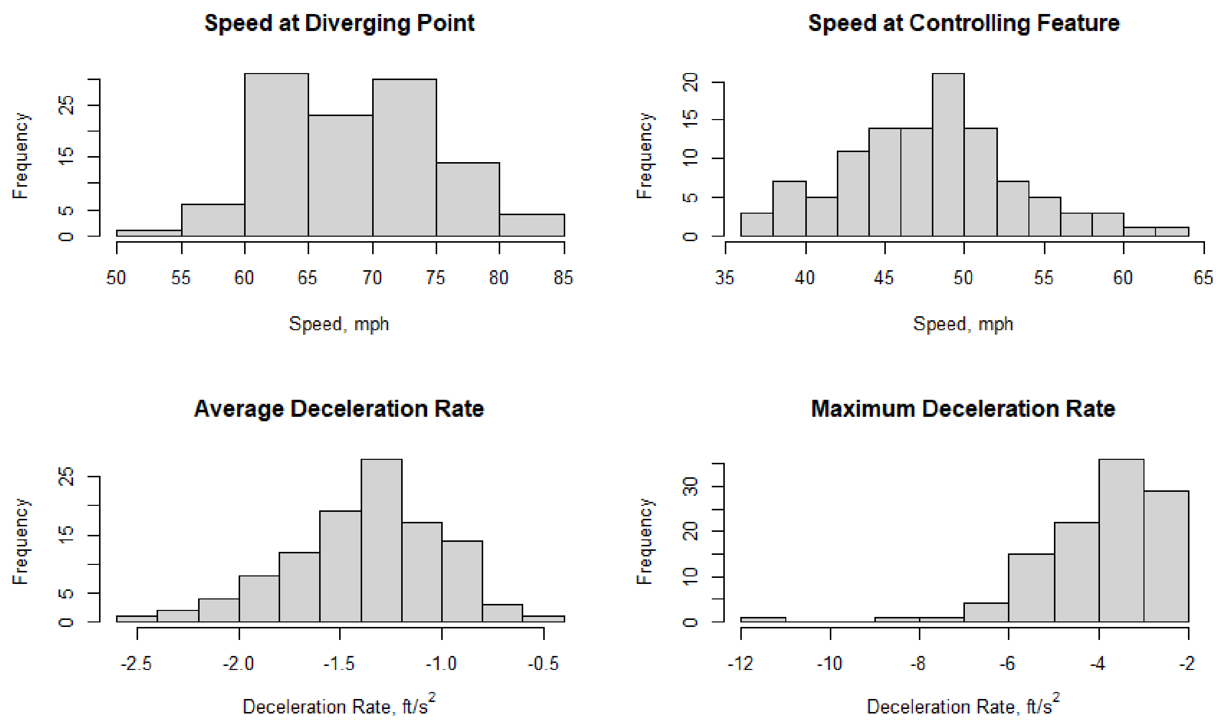


Figure 4-165. Distribution of Parameters for Site MI-11.

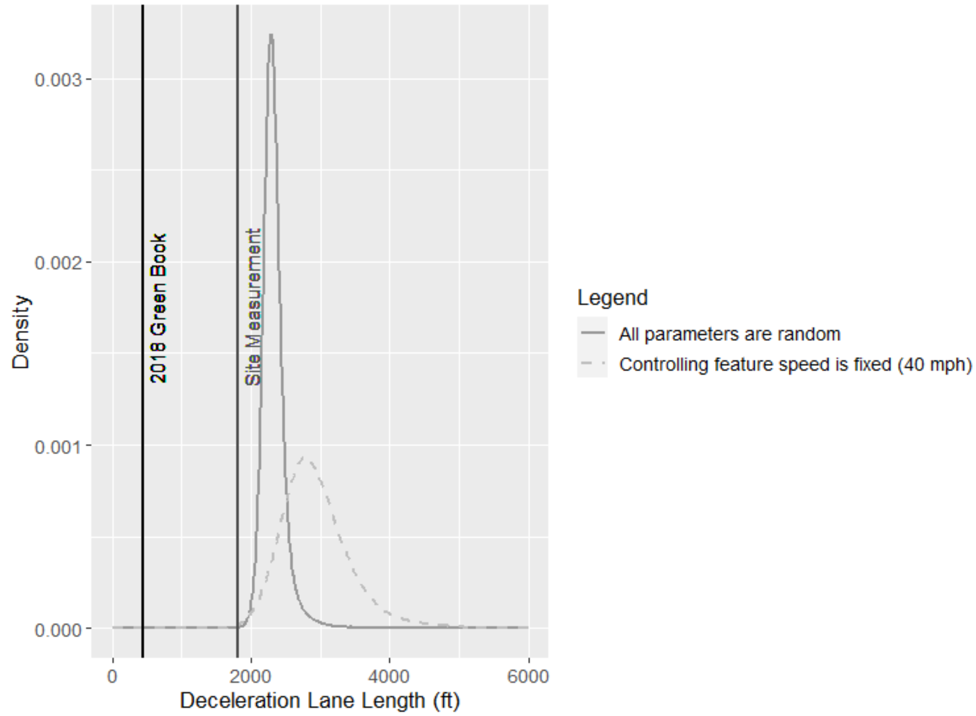


Figure 4-166. Frequency Distribution of Deceleration Lane Length based on Average Deceleration Rate for Site MI-11.

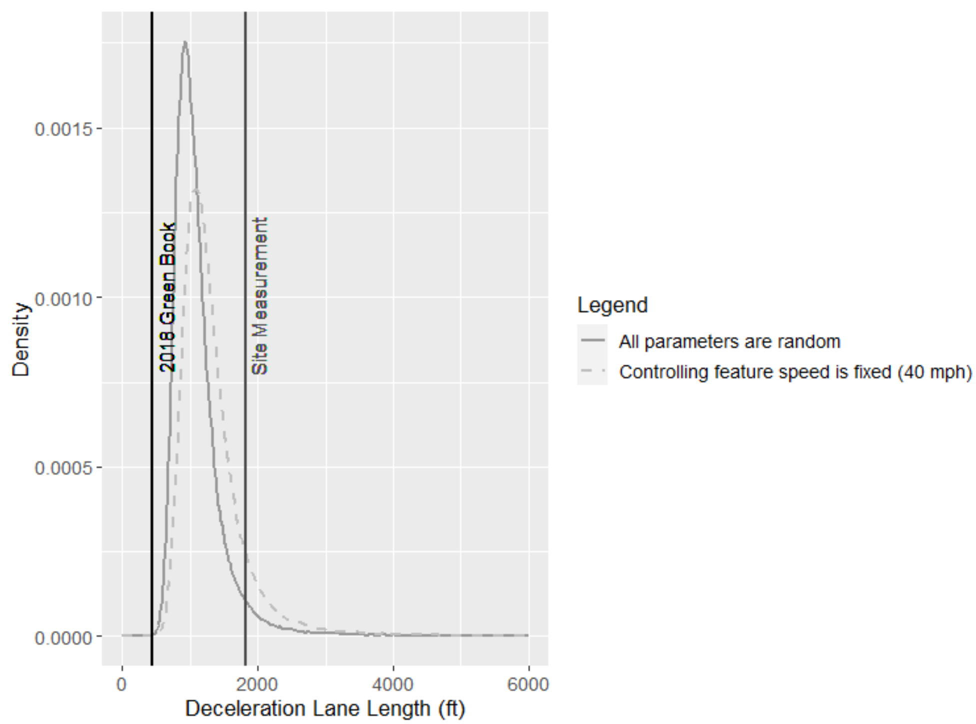


Figure 4-167. Frequency Distribution of Deceleration Lane Length based on Maximum Deceleration Rate for Site MI-11.

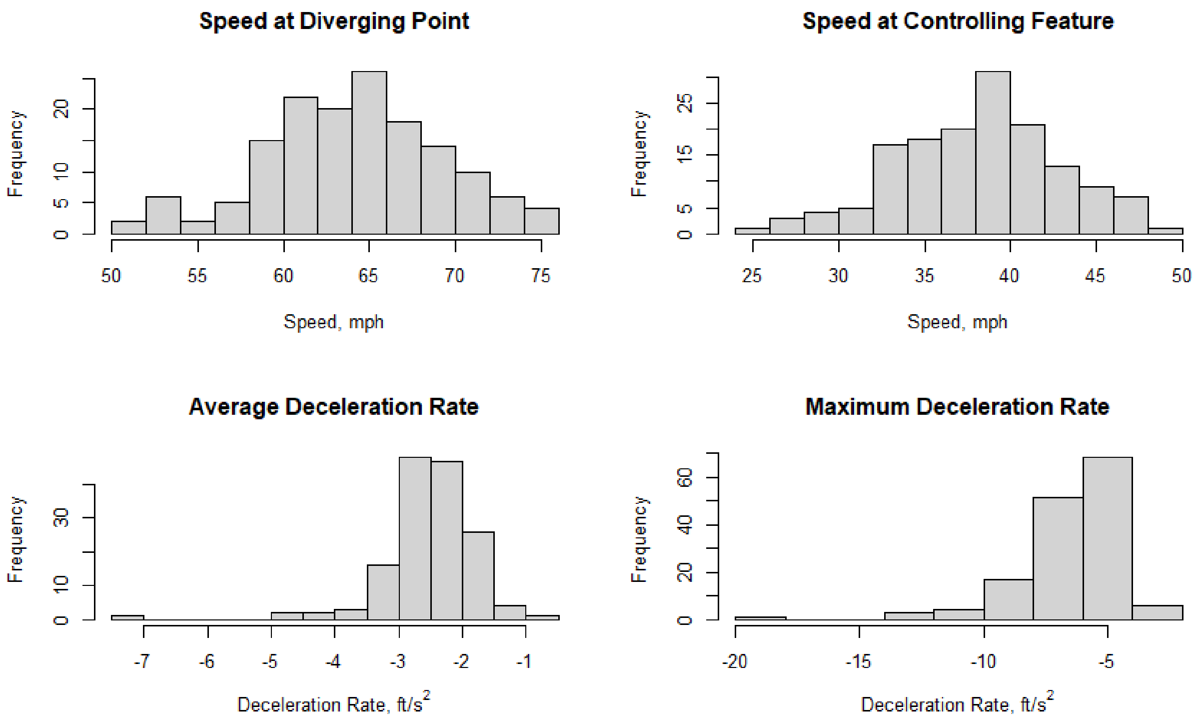


Figure 4-168. Distribution of Parameters for Site MI-12.

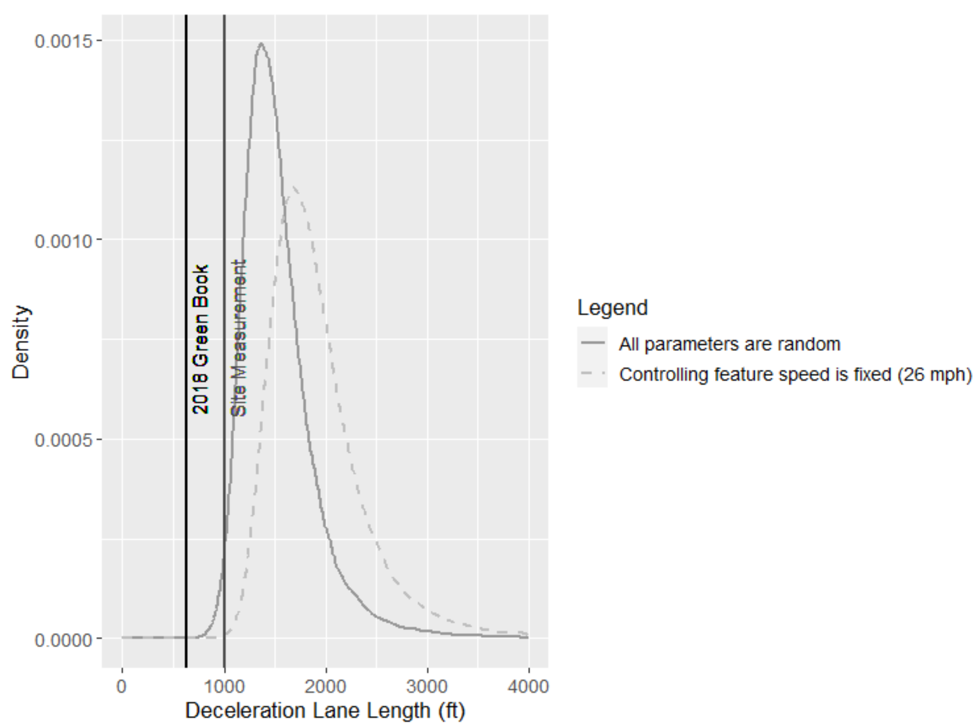


Figure 4-169. Frequency Distribution of Deceleration Lane Length based on Average Deceleration Rate for Site MI-12.

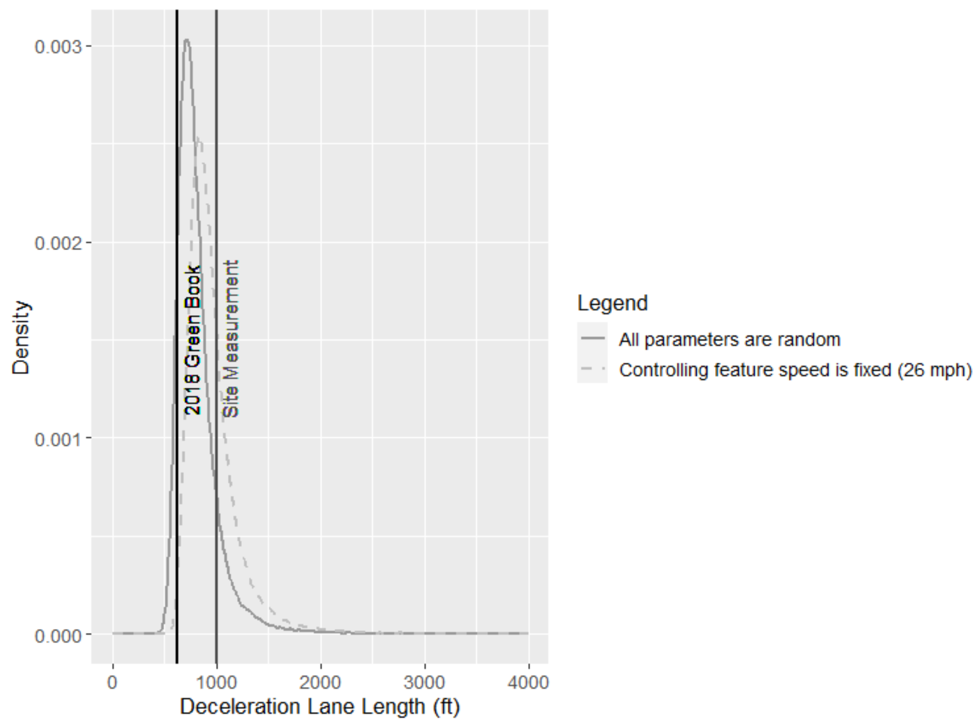


Figure 4-170. Frequency Distribution of Deceleration Lane Length based on Maximum Deceleration Rate for Site MI-12.

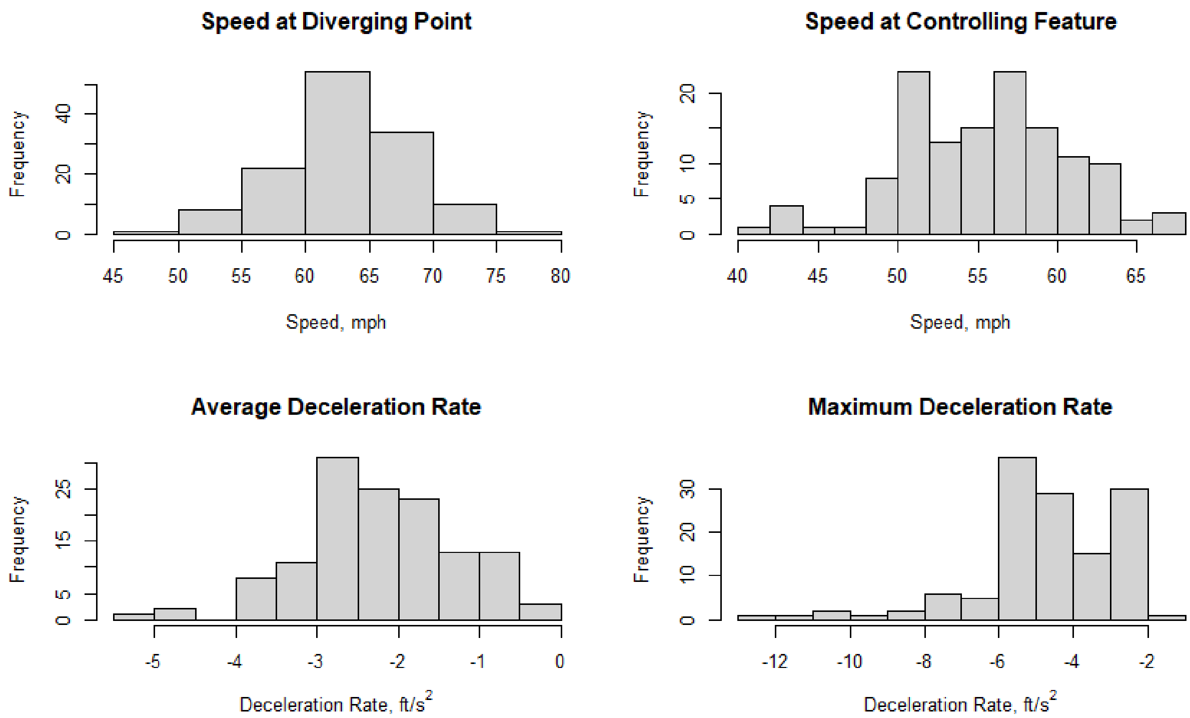


Figure 4-171. Distribution of Parameters for Site MI-13.

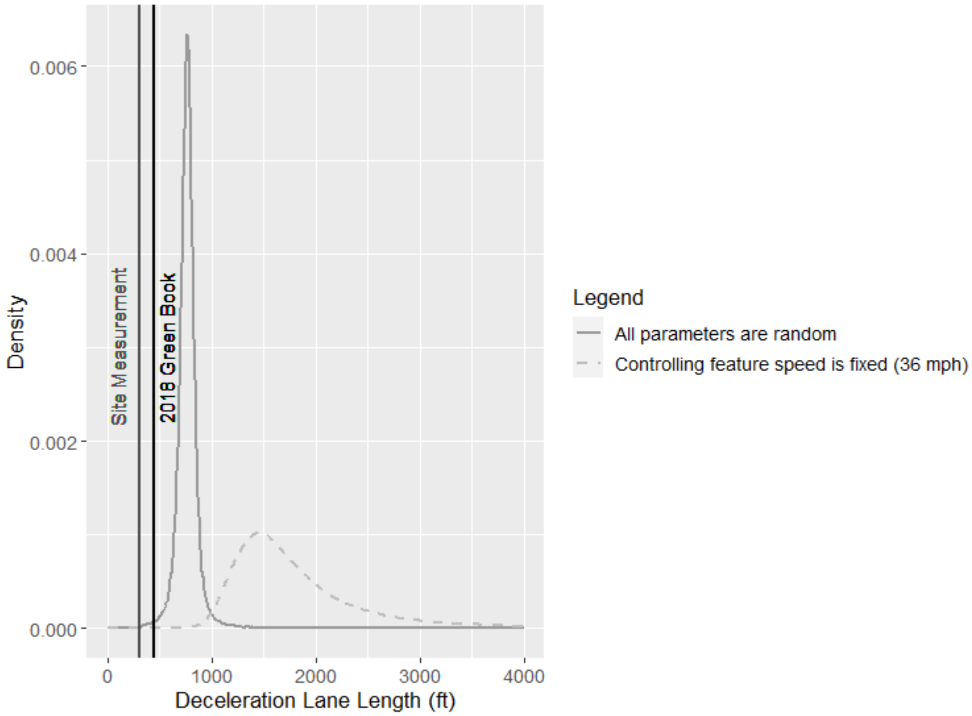


Figure 4-172. Frequency Distribution of Deceleration Lane Length based on Average Deceleration Rate for Site MI-13.

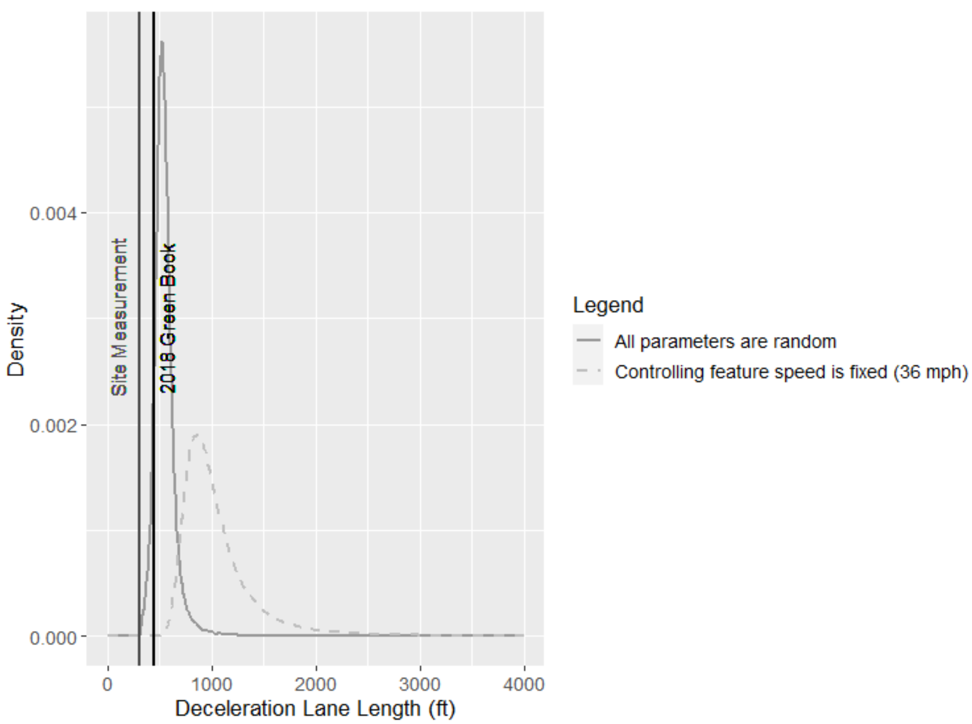


Figure 4-173. Frequency Distribution of Deceleration Lane Length based on Maximum Deceleration Rate for Site MI-13.

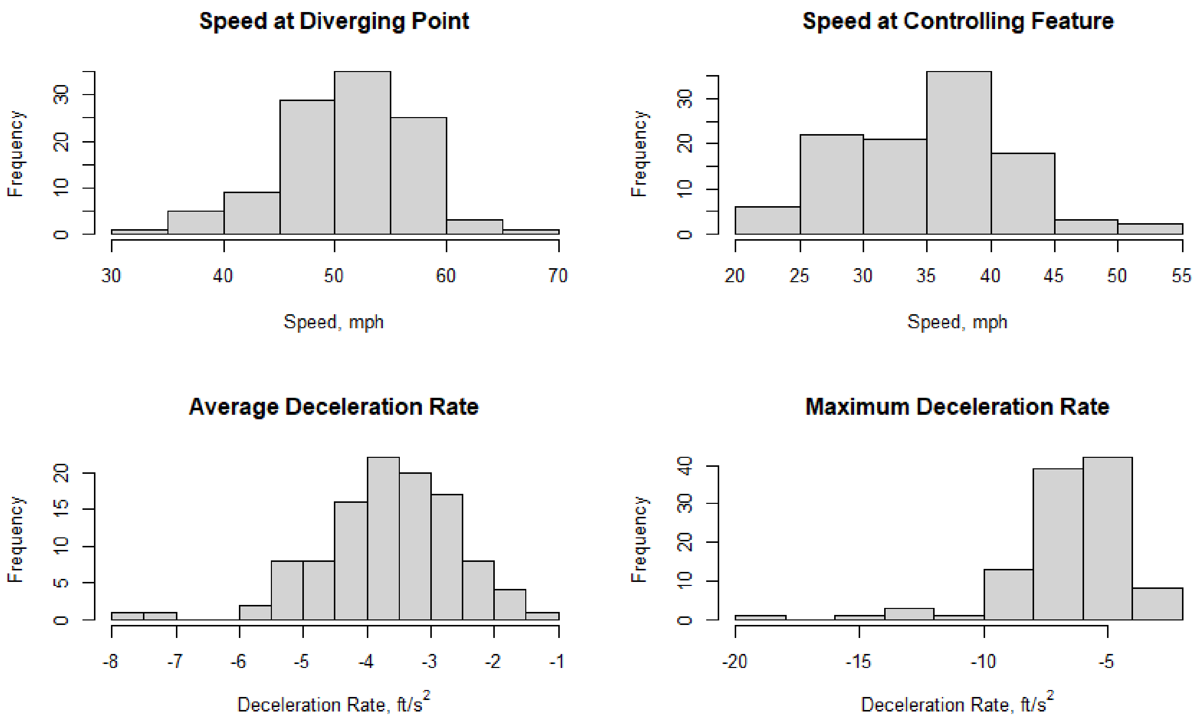


Figure 4-174. Distribution of Parameters for Site MI-14.

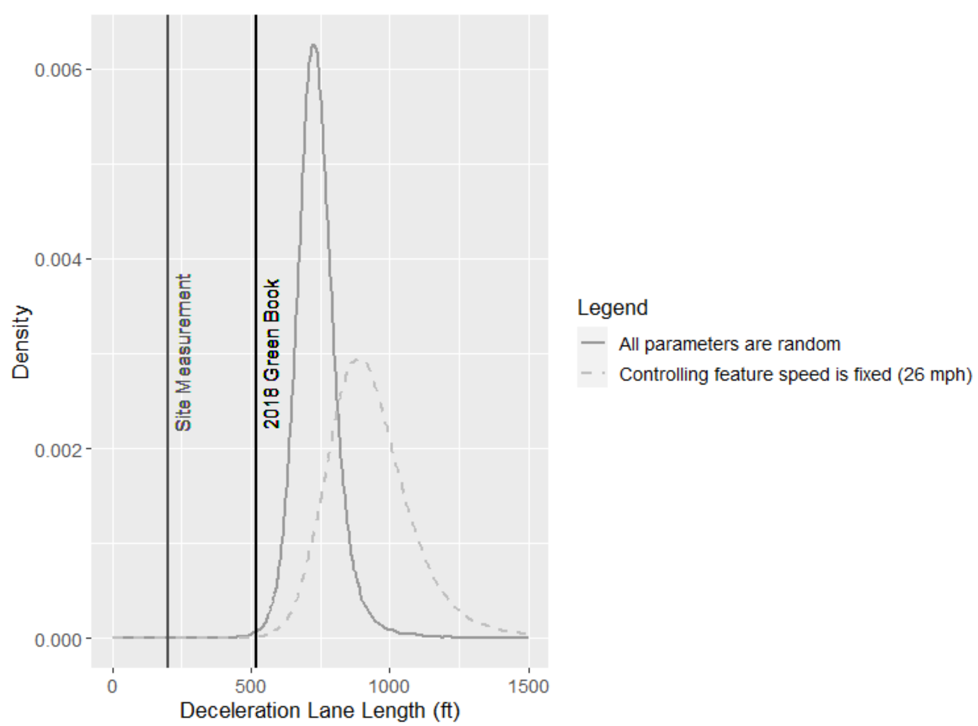


Figure 4-175. Frequency Distribution of Deceleration Lane Length based on Average Deceleration Rate for Site MI-14.

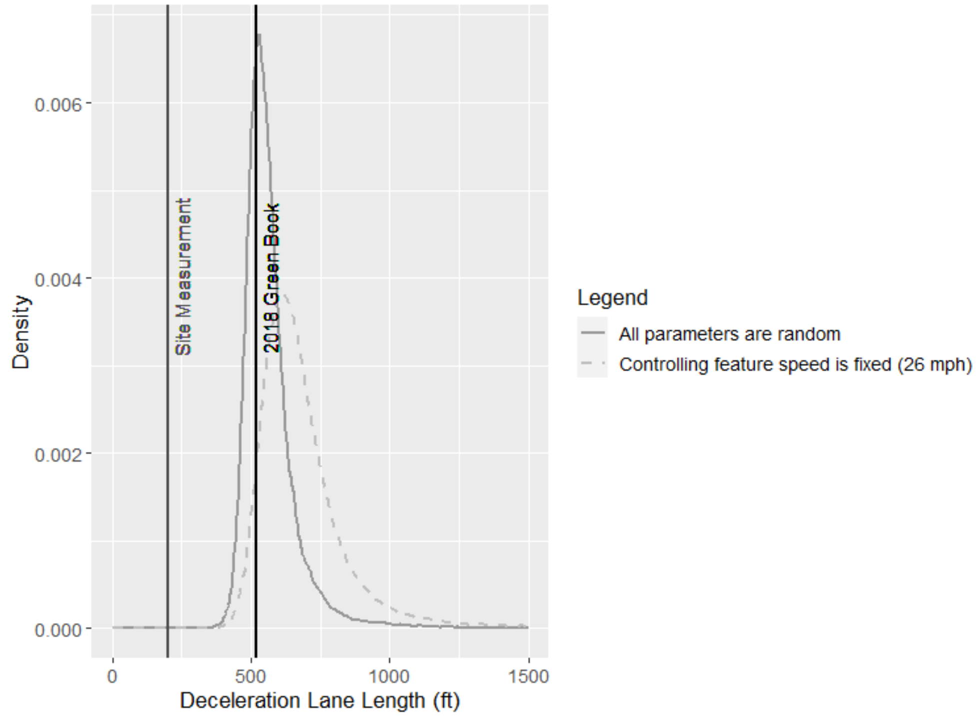


Figure 4-176. Frequency Distribution of Deceleration Lane Length based on Maximum Deceleration Rate for Site MI-14.

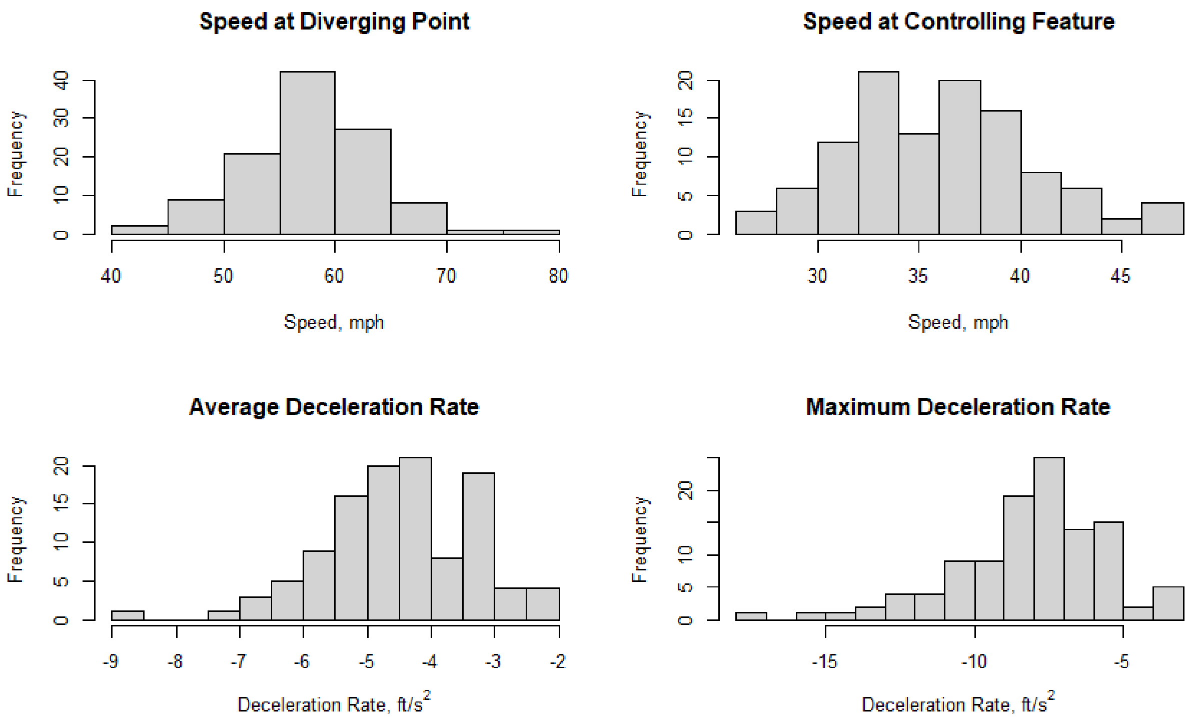


Figure 4-177. Distribution of Parameters for Site MI-15.

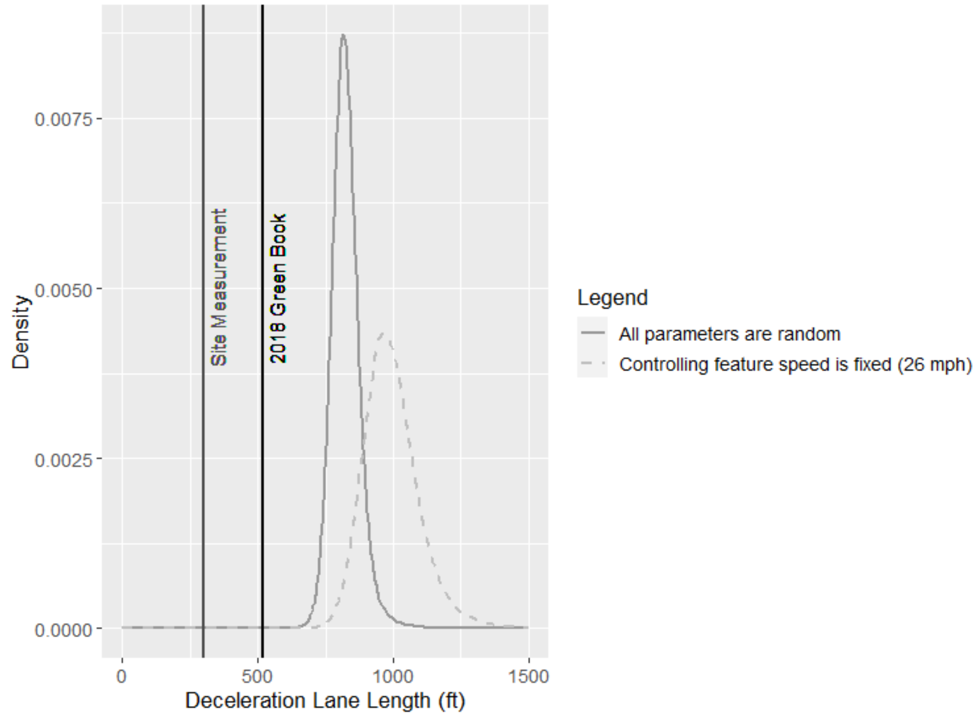


Figure 4-178. Frequency Distribution of Deceleration Lane Length based on Average Deceleration Rate for Site MI-15.

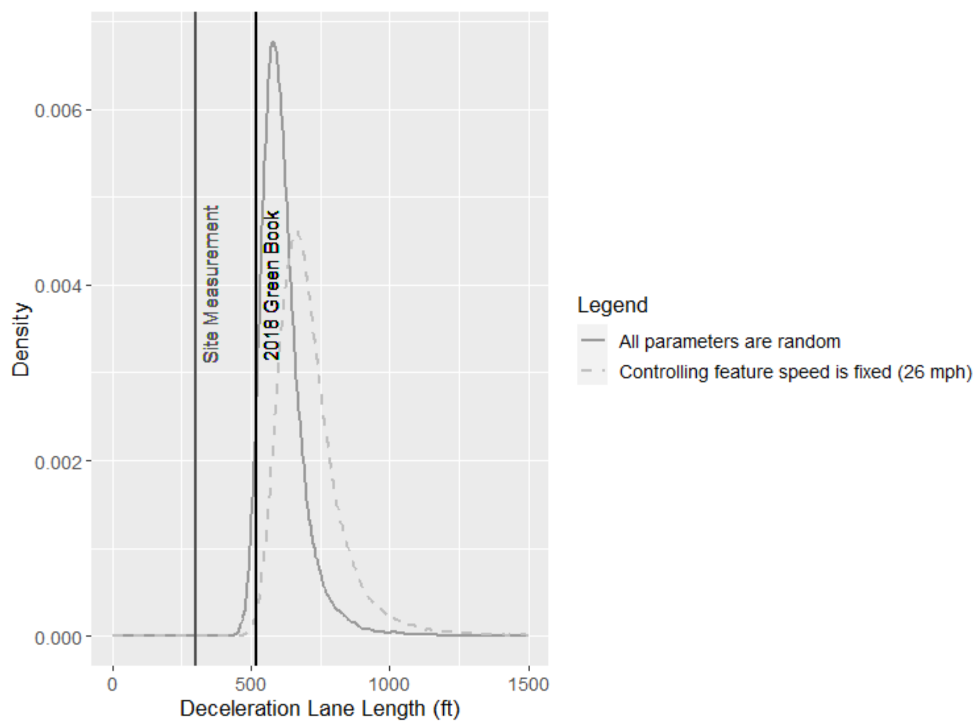


Figure 4-179. Frequency Distribution of Deceleration Lane Length based on Maximum Deceleration Rate for Site MI-15.

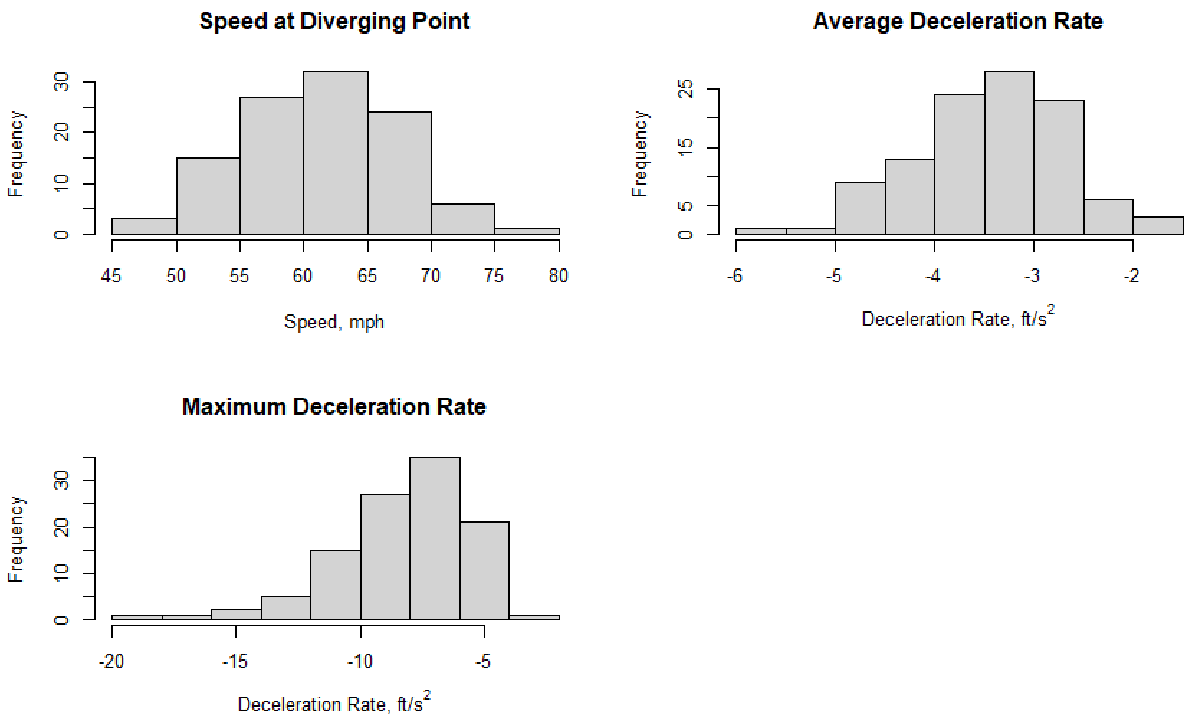


Figure 4-180. Distribution of Parameters for Site MI-16.

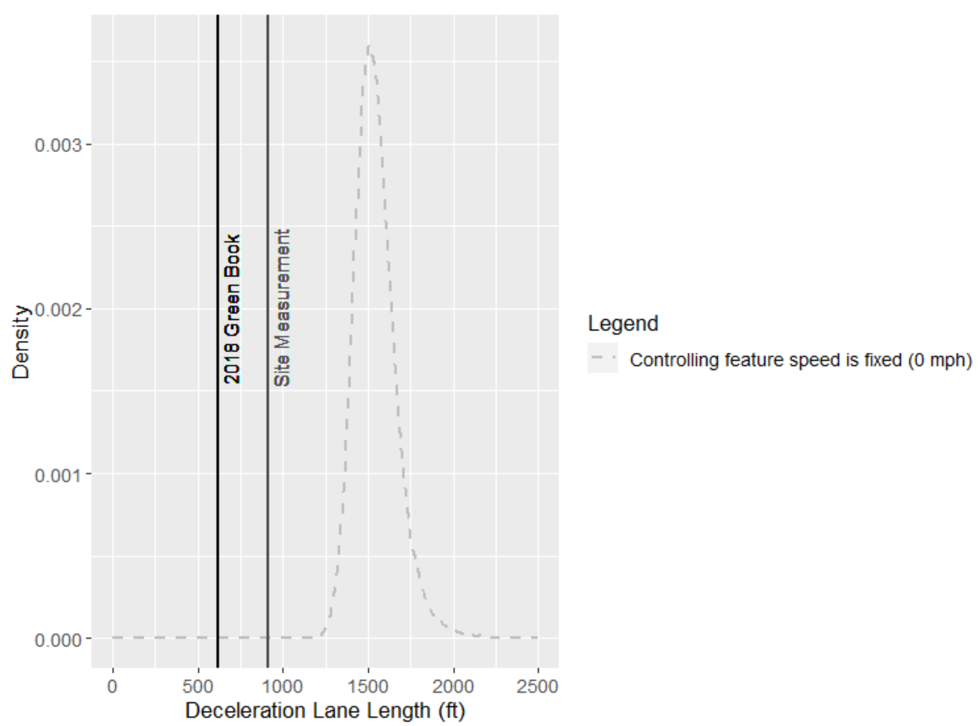


Figure 4-181. Frequency Distribution of Deceleration Lane Length based on Average Deceleration Rate for Site MI-16.

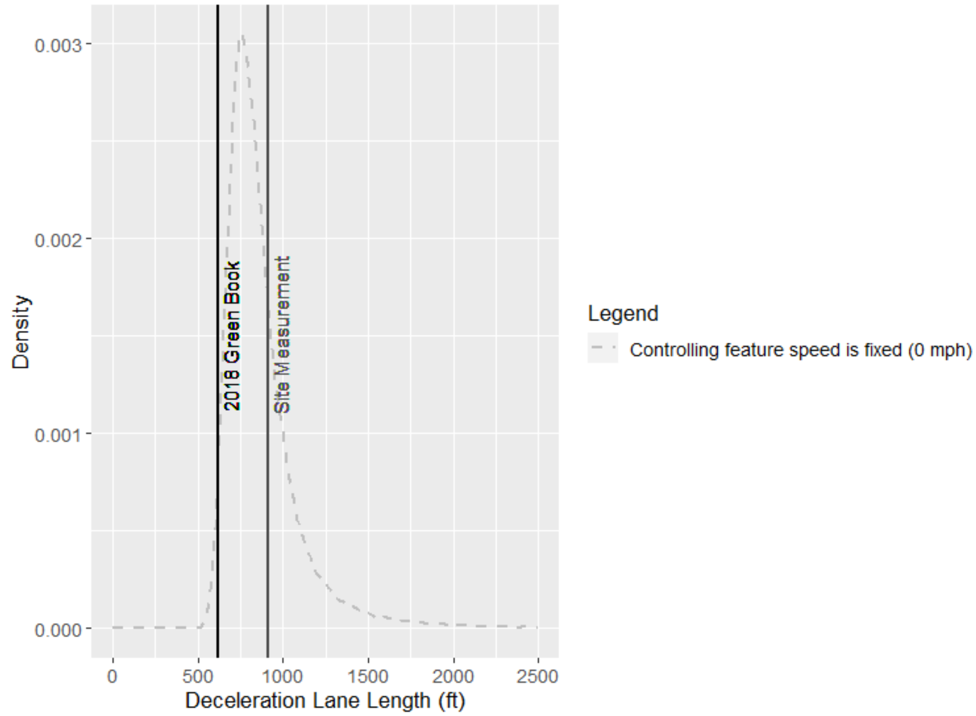


Figure 4-182. Frequency Distribution of Deceleration Lane Length based on Maximum Deceleration Rate for Site MI-16.

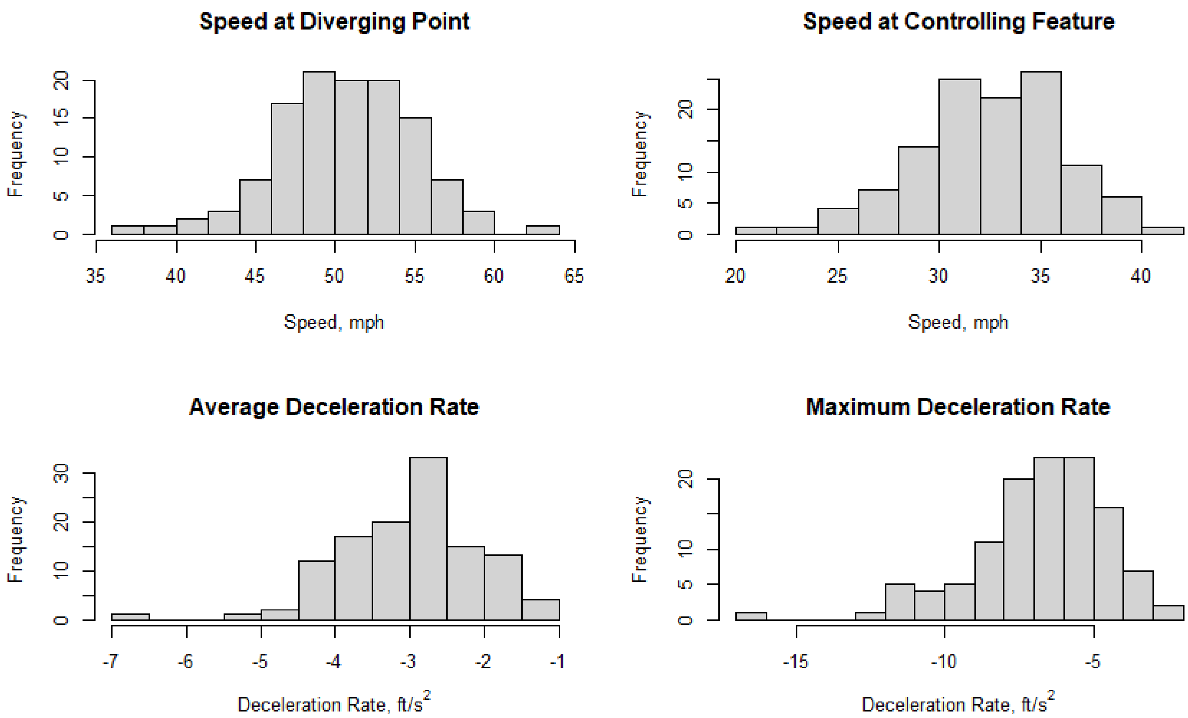


Figure 4-183. Distribution of Parameters for Site NC-9.

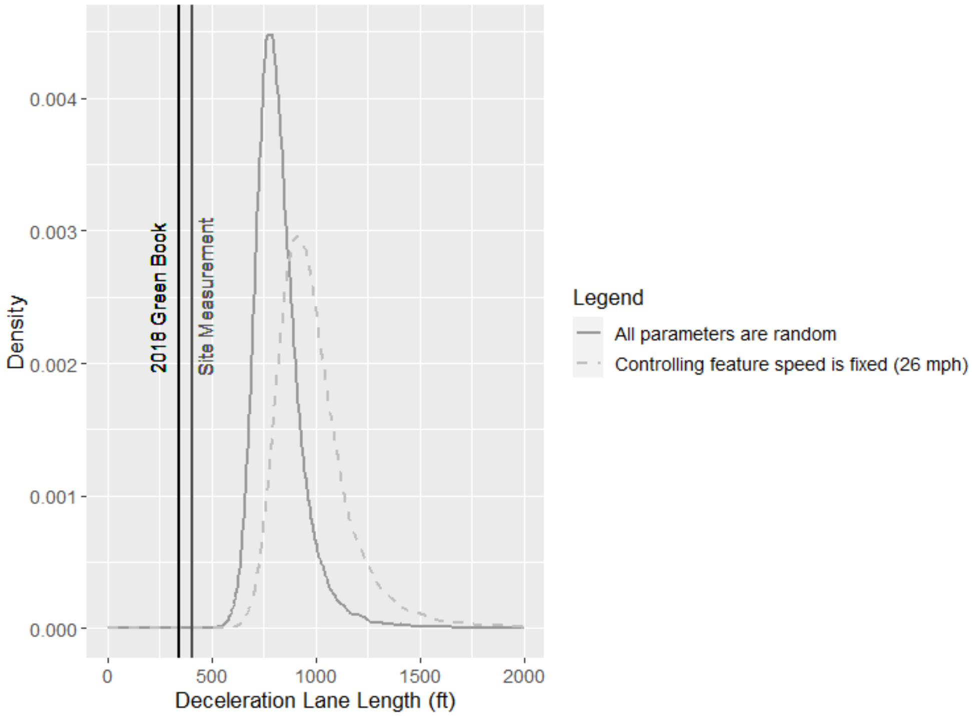


Figure 4-184. Frequency Distribution of Deceleration Lane Length based on Average Deceleration Rate for Site NC-9.

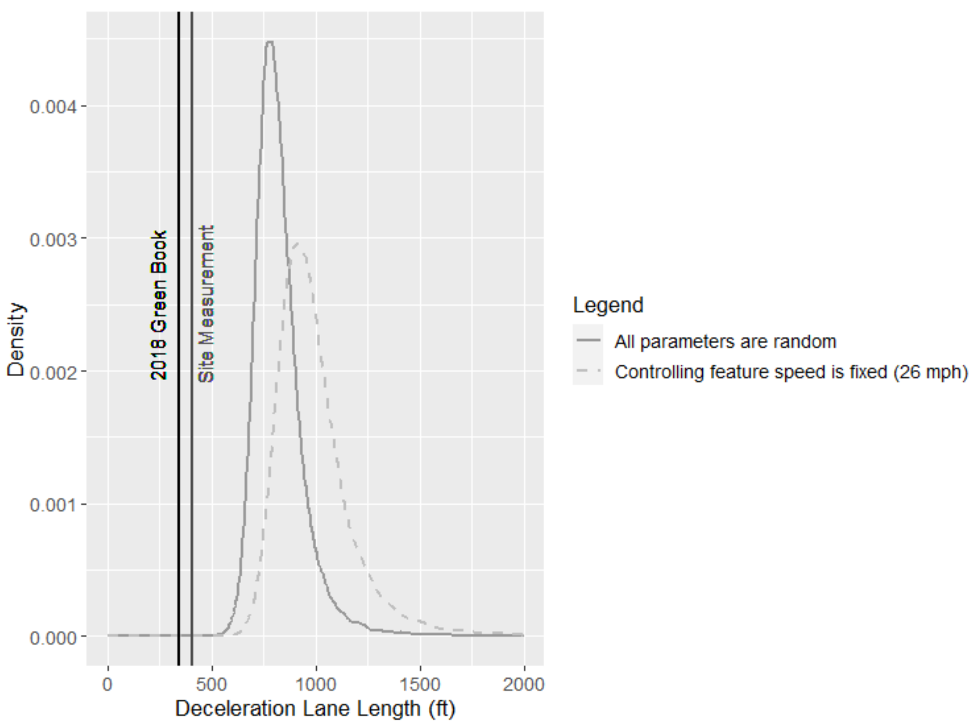


Figure 4-185. Frequency Distribution of Deceleration Lane Length based on Maximum Deceleration Rate for Site NC-9.

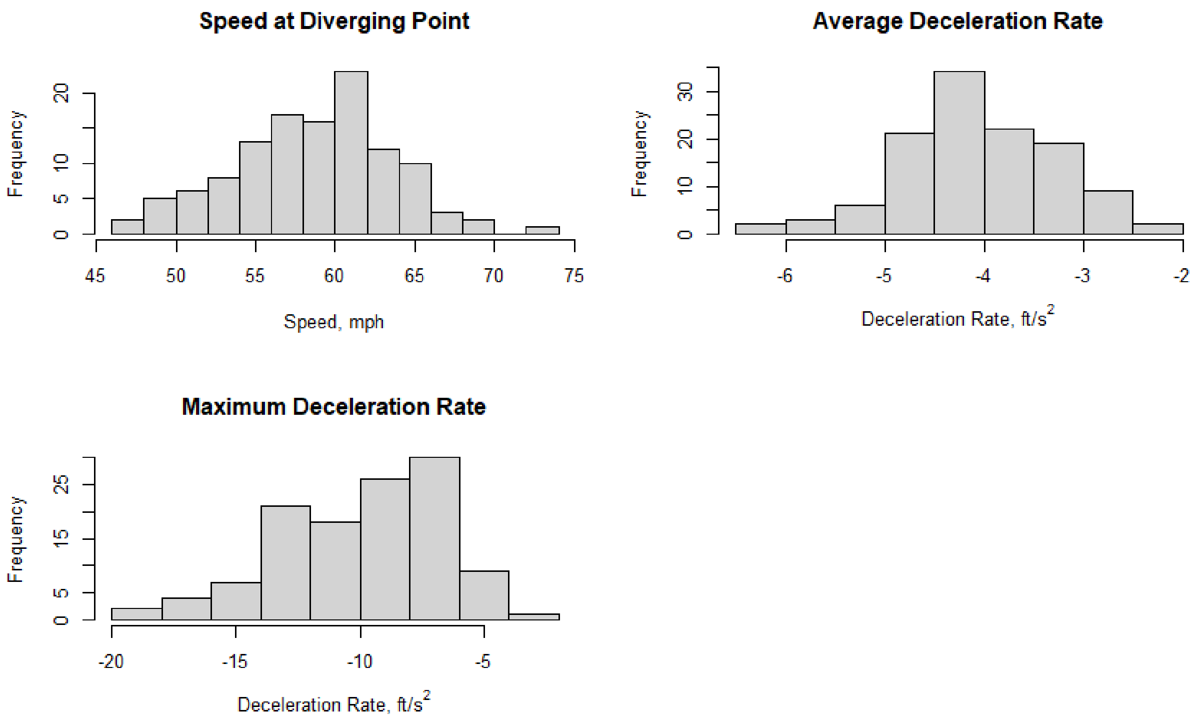


Figure 4-186. Distribution of Parameters for Site NC-11.

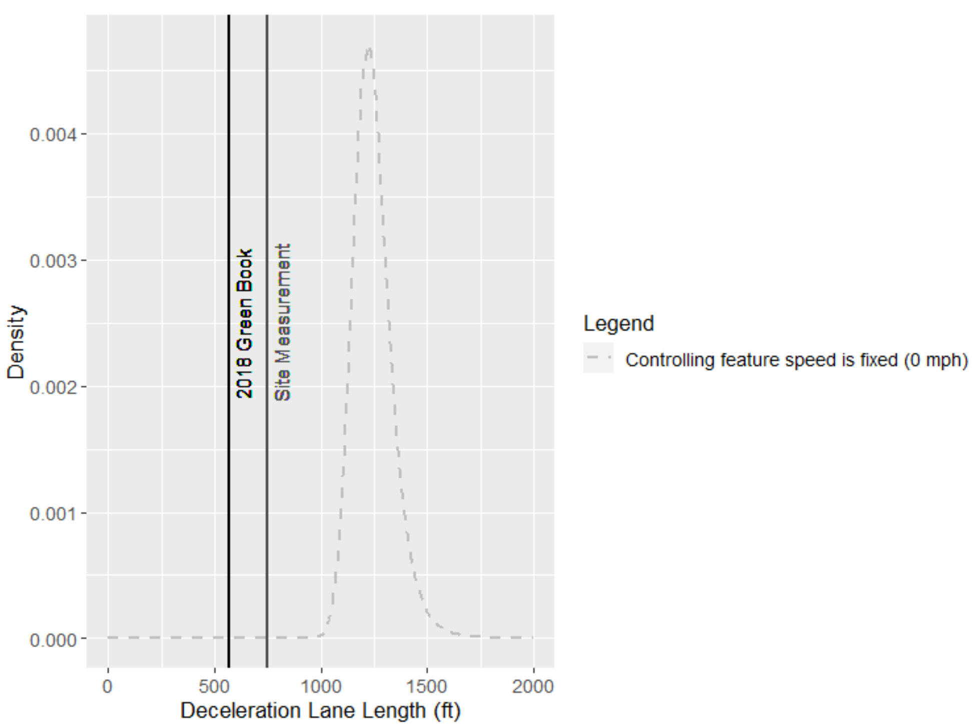


Figure 4-187. Frequency Distribution of Deceleration Lane Length based on Average Deceleration Rate for Site NC-11.

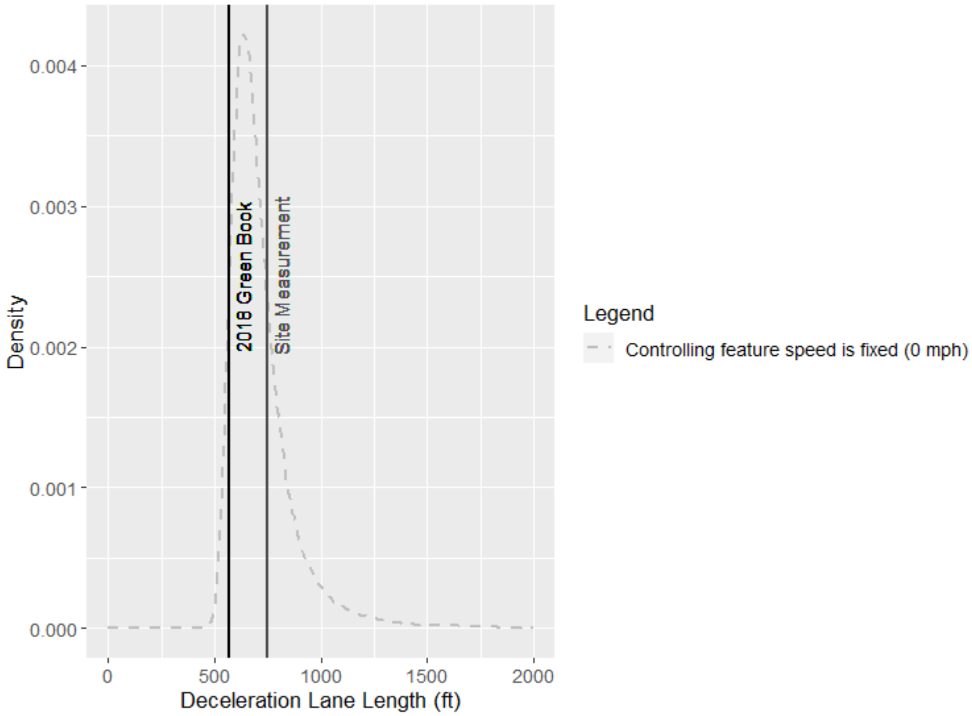


Figure 4-188. Frequency Distribution of Deceleration Lane Length based on Maximum Deceleration Rate for Site NC-11.

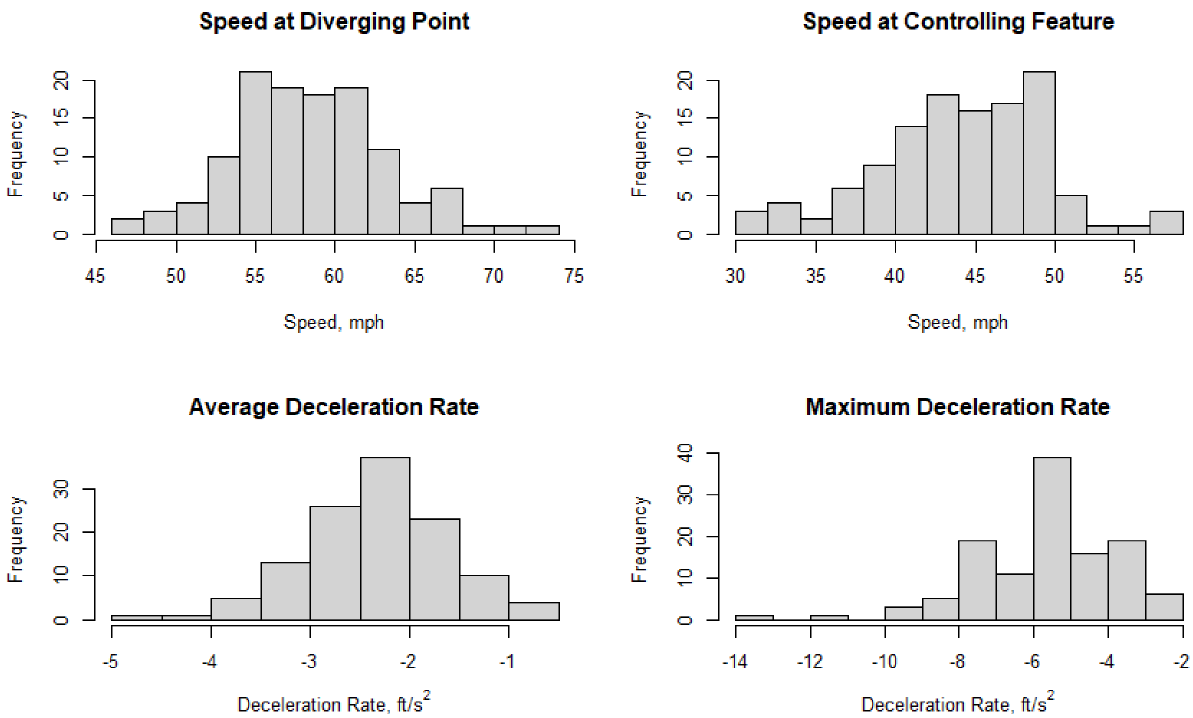


Figure 4-189. Distribution of Parameters for Site NC-12.

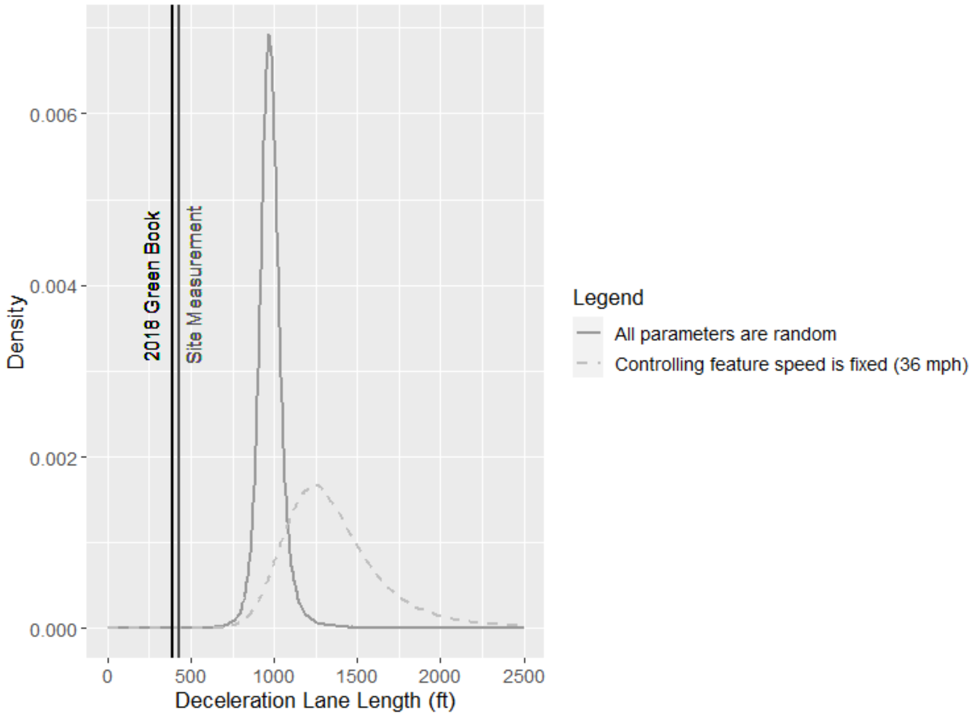


Figure 4-190. Frequency Distribution of Deceleration Lane Length based on Average Deceleration Rate for Site NC-12.

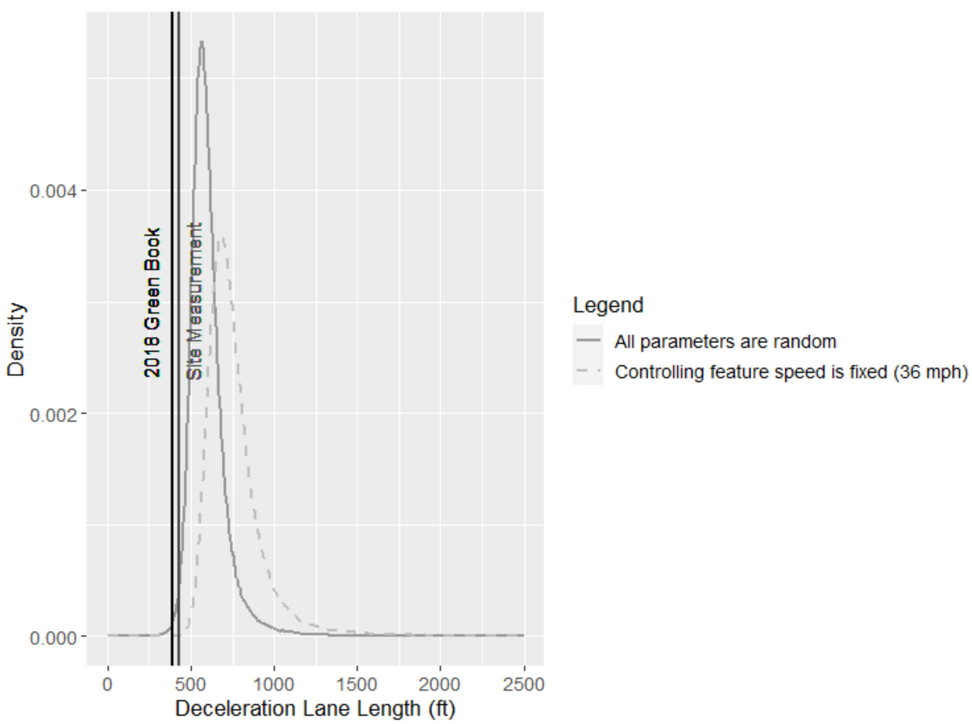


Figure 4-191. Frequency Distribution of Deceleration Lane Length based on Maximum Deceleration Rate for Site NC-12.

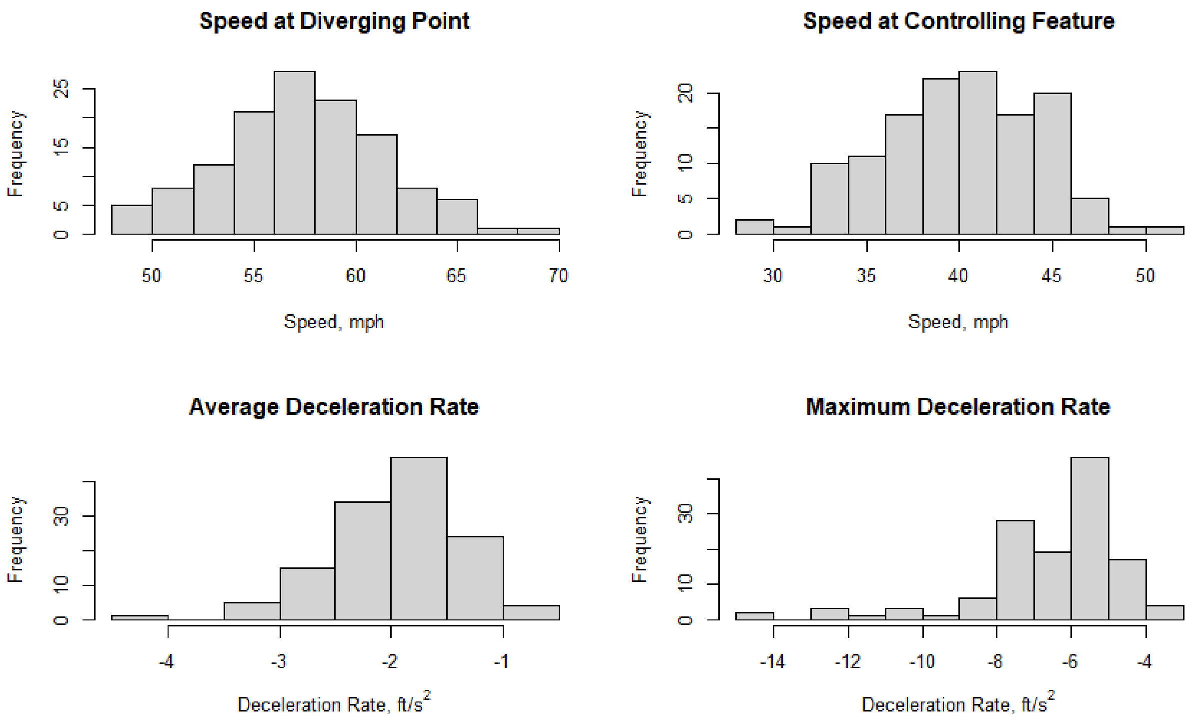


Figure 4-192. Distribution of Parameters for Site NC-13.

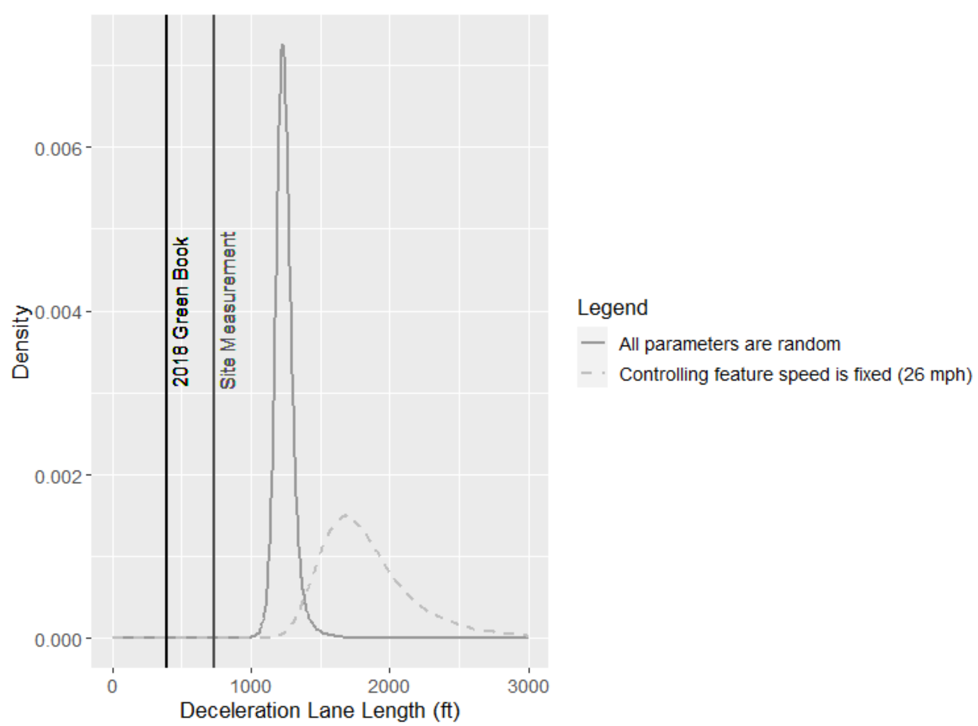


Figure 4-193. Frequency Distribution of Deceleration Lane Length based on Average Deceleration Rate for Site NC-13.

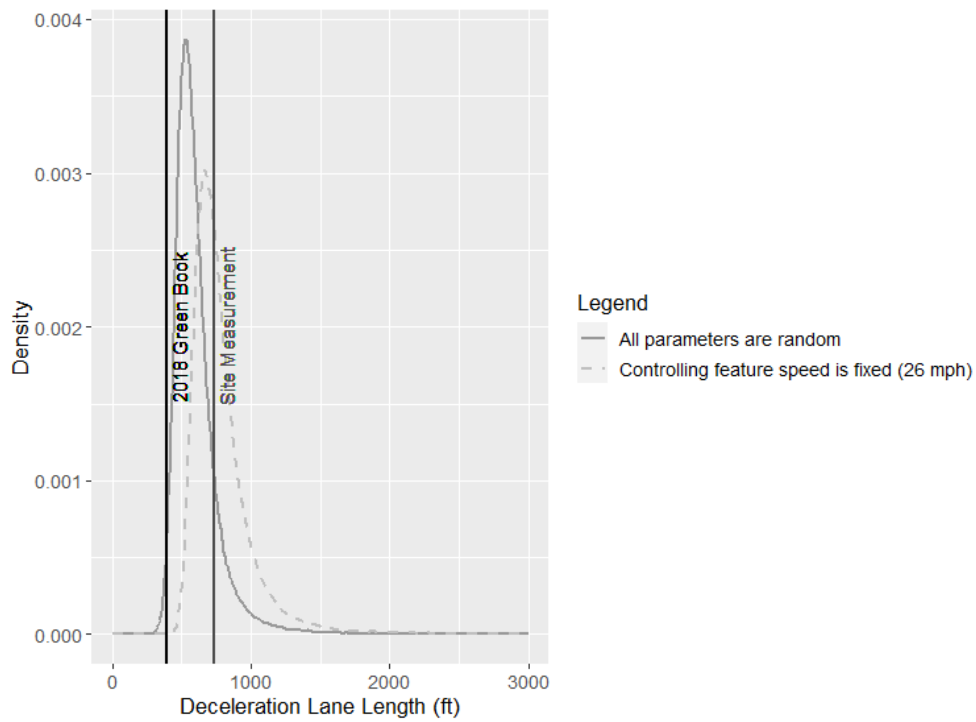


Figure 4-194. Frequency Distribution of Deceleration Lane Length based on Maximum Deceleration Rate for Site NC-13.

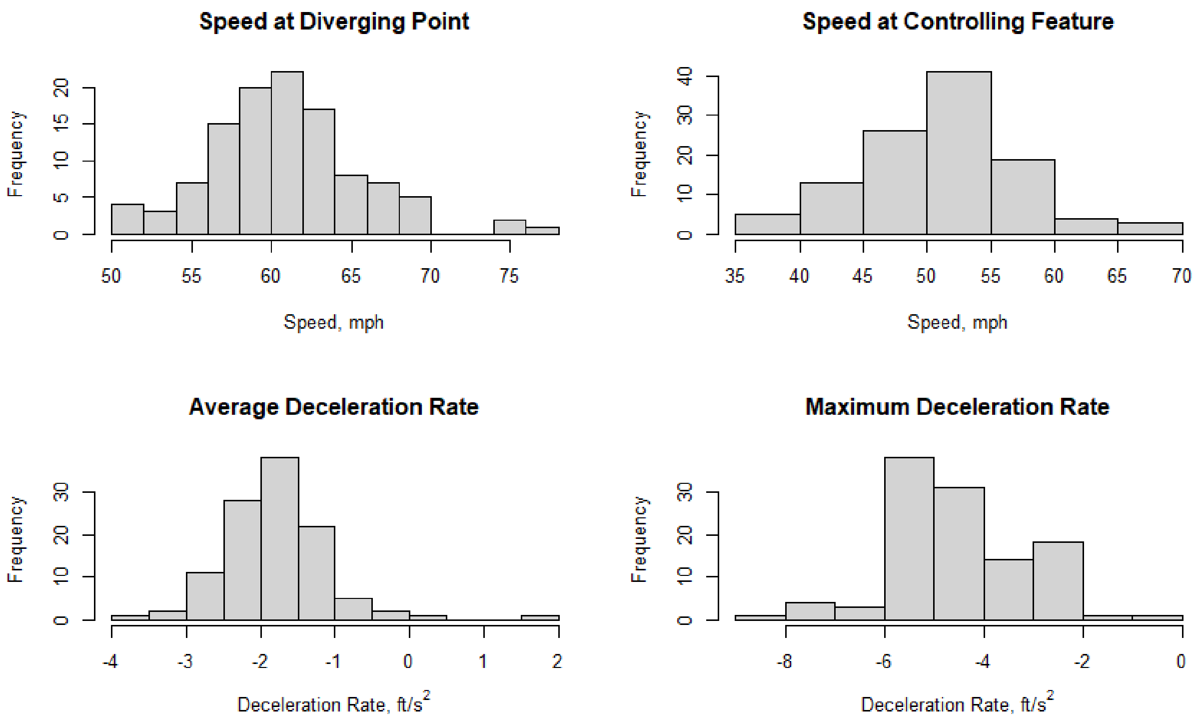


Figure 4-195. Distribution of Parameters for Site NC-14.

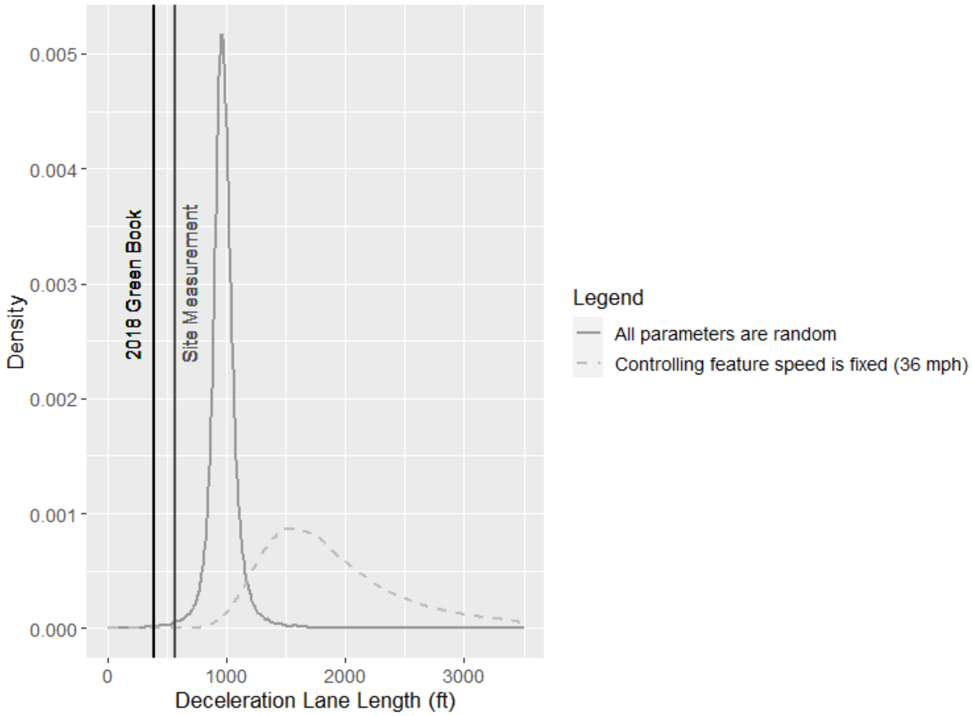


Figure 4-196. Frequency Distribution of Deceleration Lane Length based on Average Deceleration Rate for Site NC-14.

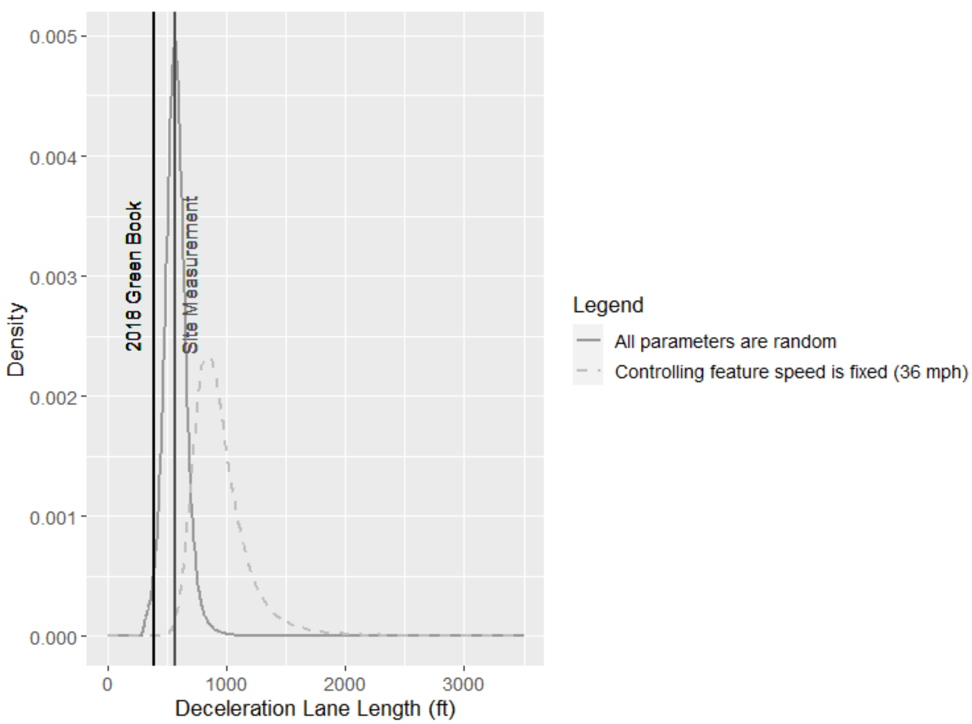


Figure 4-197. Frequency Distribution of Deceleration Lane Length based on Maximum Deceleration Rate for Site NC-14.

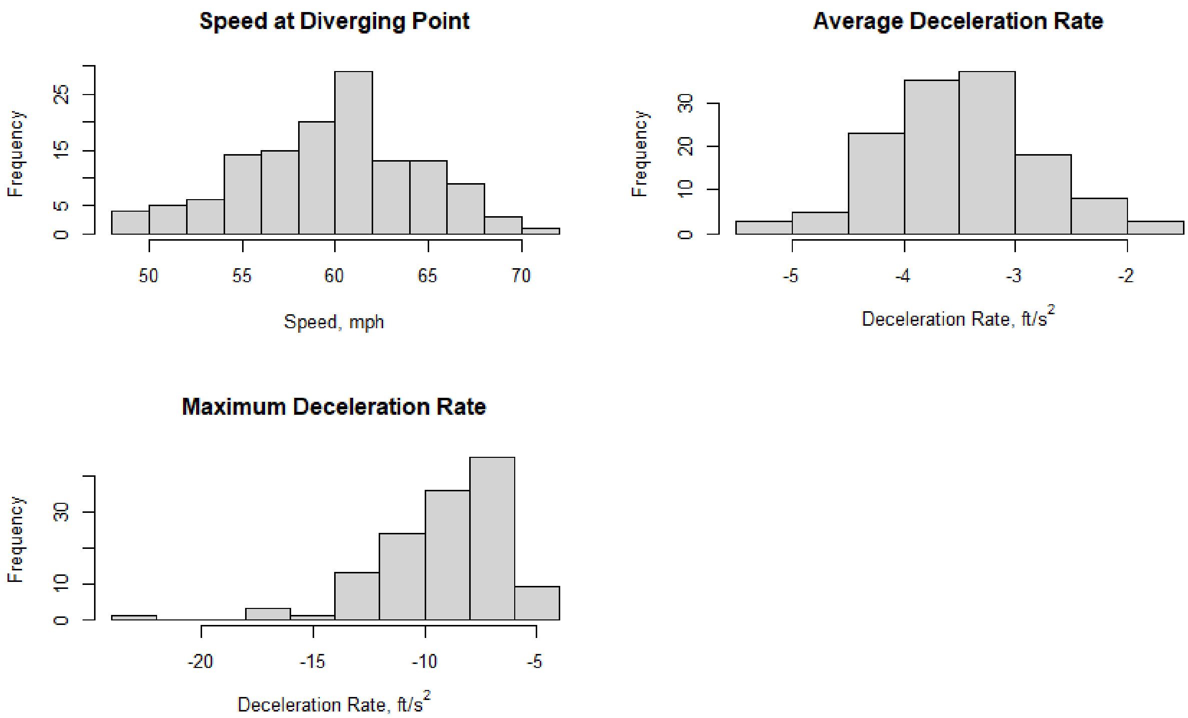


Figure 4-198. Distribution of Parameters for Site NC-15.

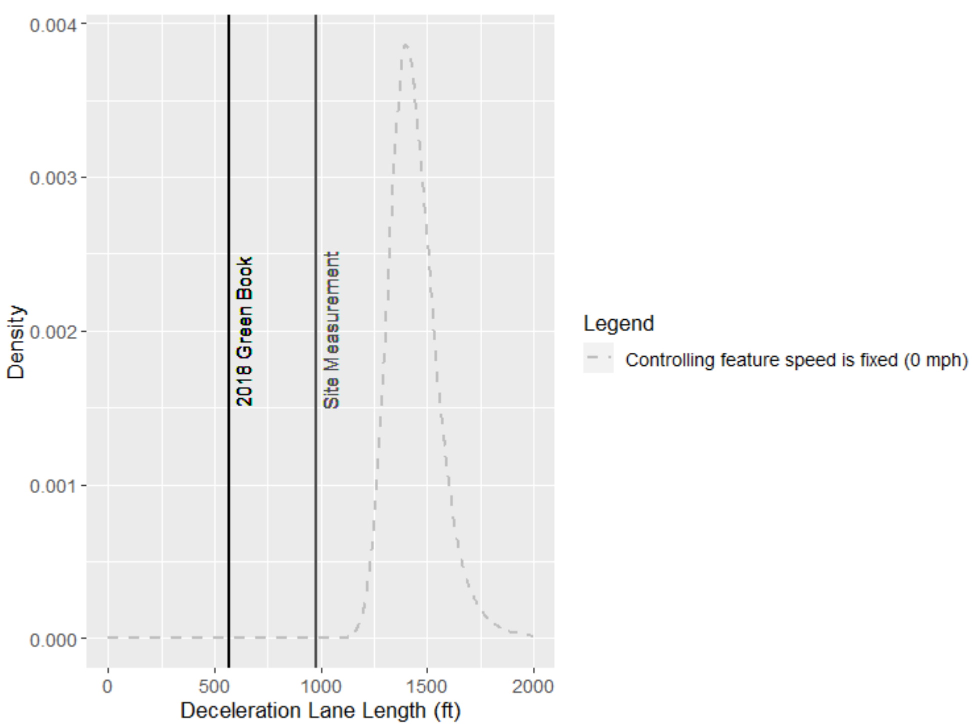


Figure 4-199. Frequency Distribution of Deceleration Lane Length based on Average Deceleration Rate for Site NC-15.

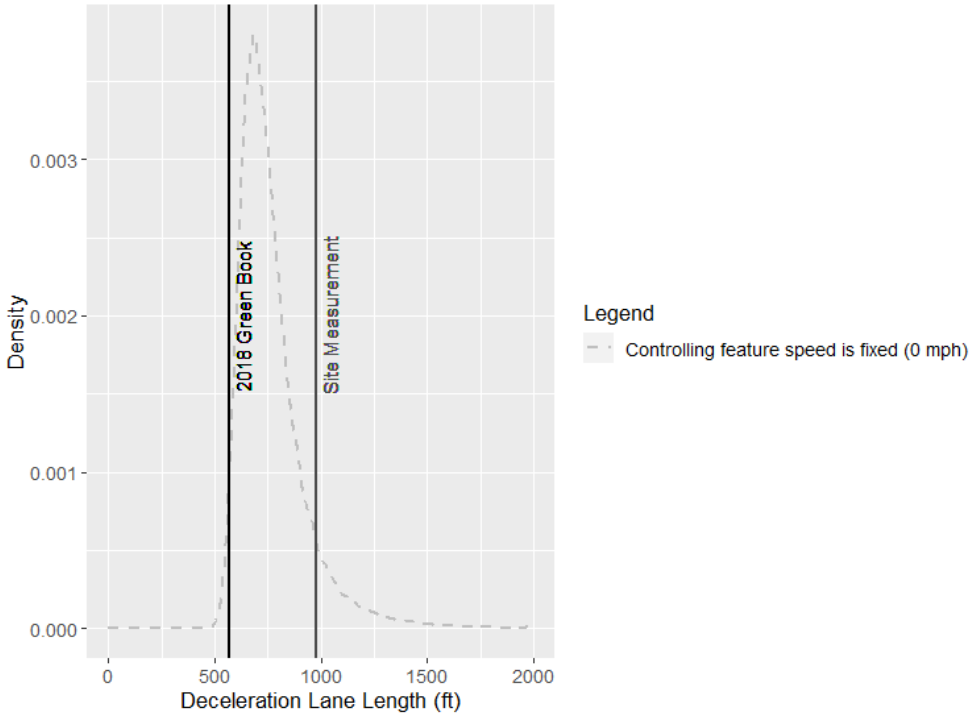


Figure 4-200. Frequency Distribution of Deceleration Lane Length based on Maximum Deceleration Rate for Site NC-15.

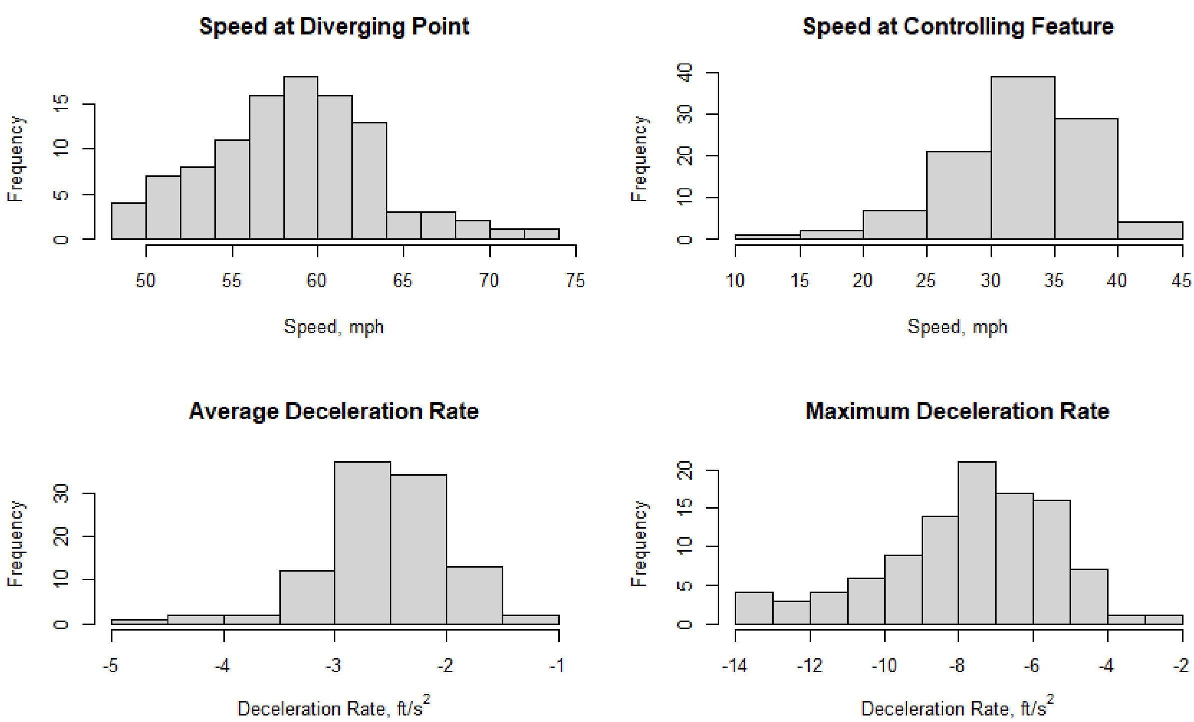


Figure 4-201. Distribution of Parameters for Site NC-16.

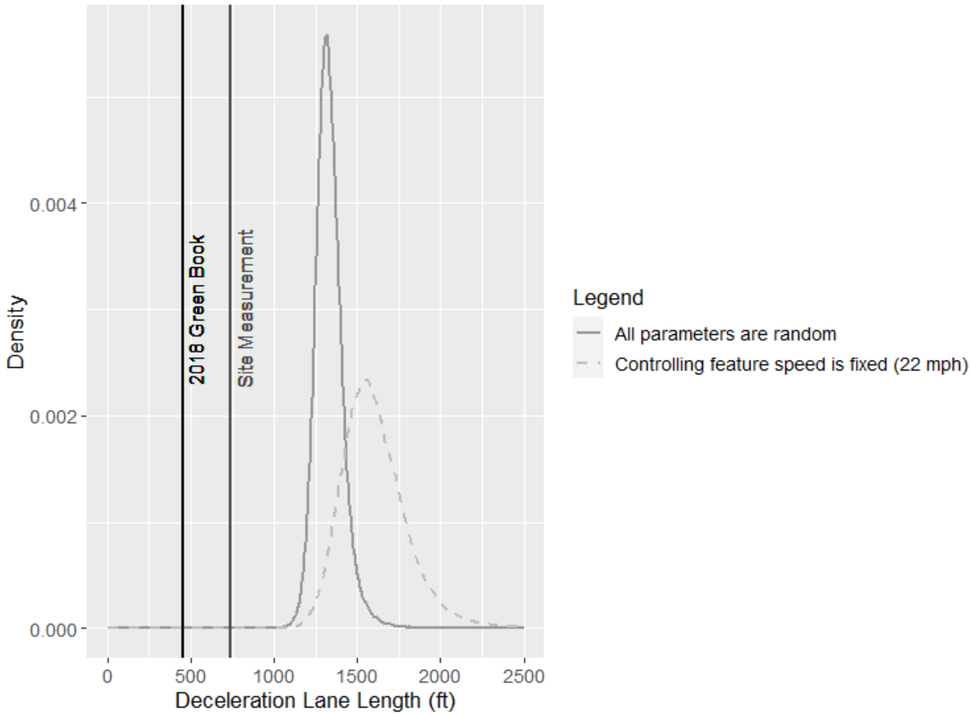


Figure 4-202. Frequency Distribution of Deceleration Lane Length based on Average Deceleration Rate for Site NC-16.

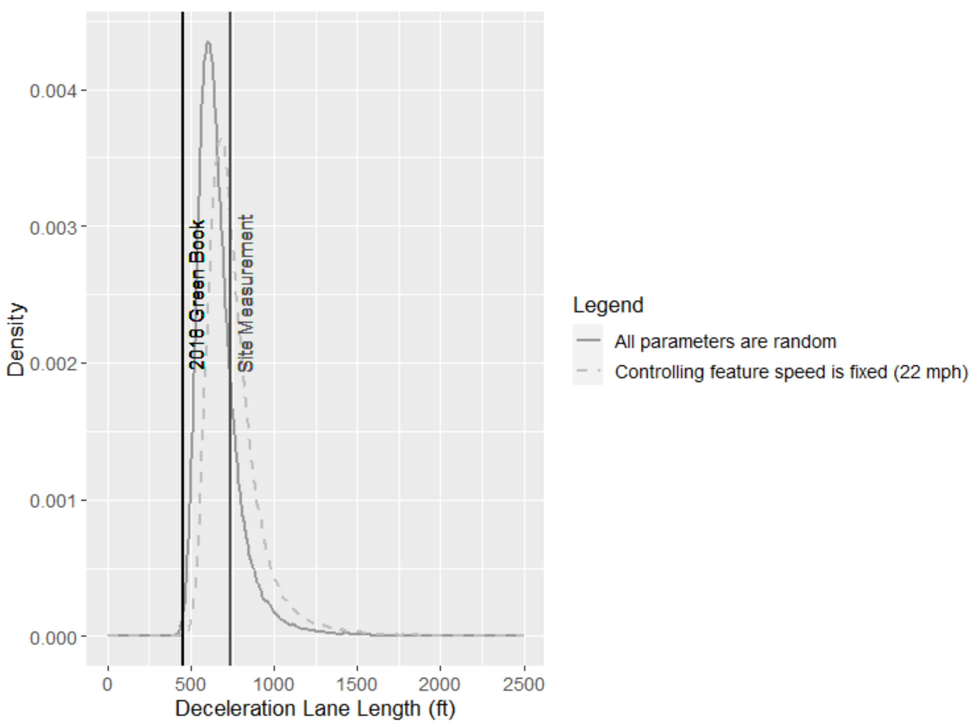


Figure 4-203. Frequency Distribution of Deceleration Lane Length based on Maximum Deceleration Rate for Site NC-16.

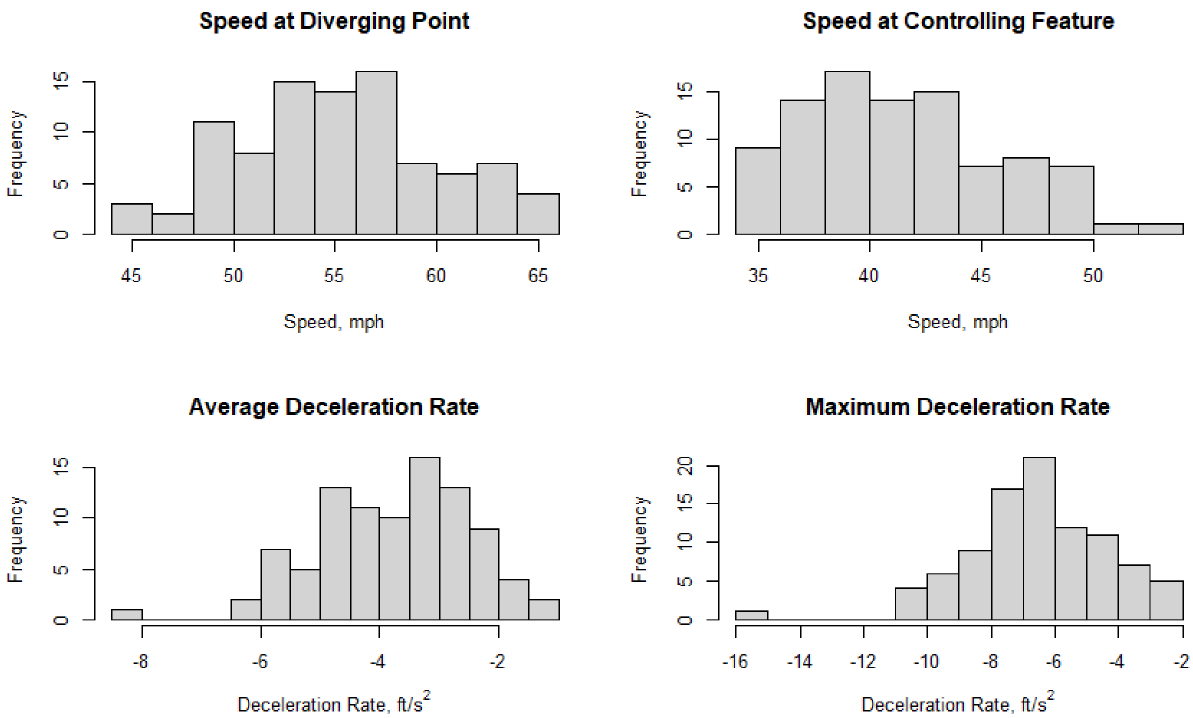


Figure 4-204. Distribution of Parameters for Site PA-10.

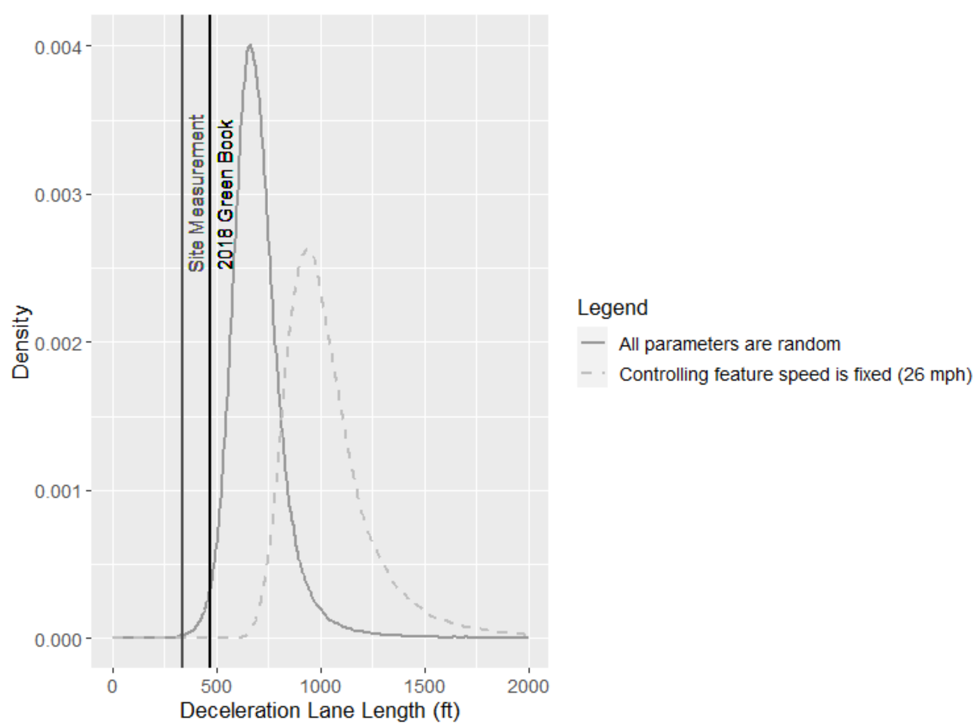


Figure 4-205. Frequency Distribution of Deceleration Lane Length based on Average Deceleration Rate for Site PA-10.

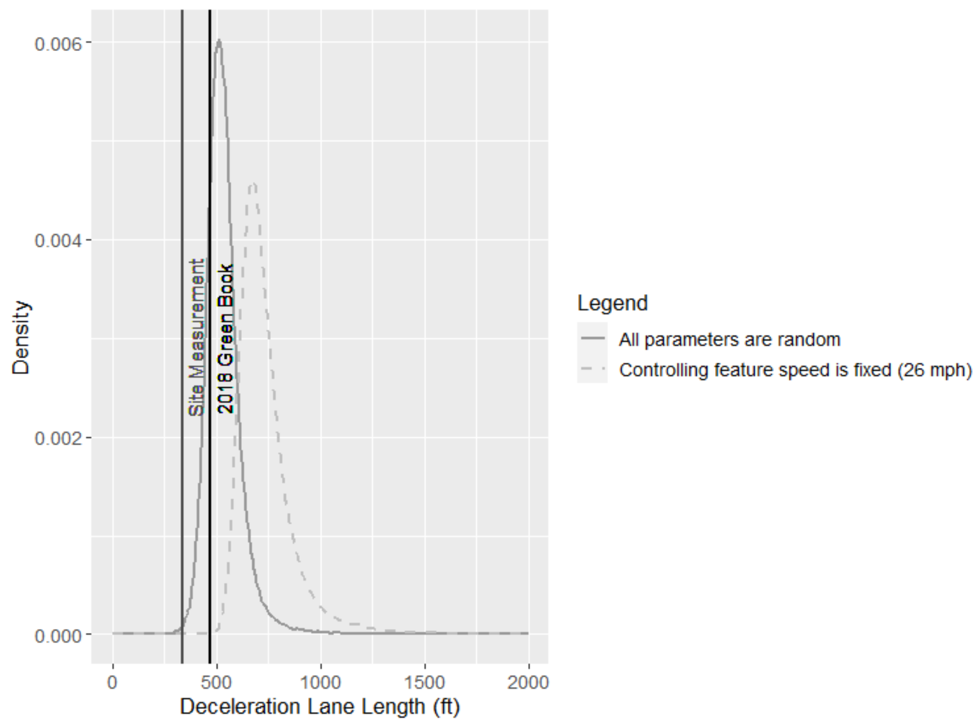


Figure 4-206. Frequency Distribution of Deceleration Lane Length based on Maximum Deceleration Rate for Site PA-10.

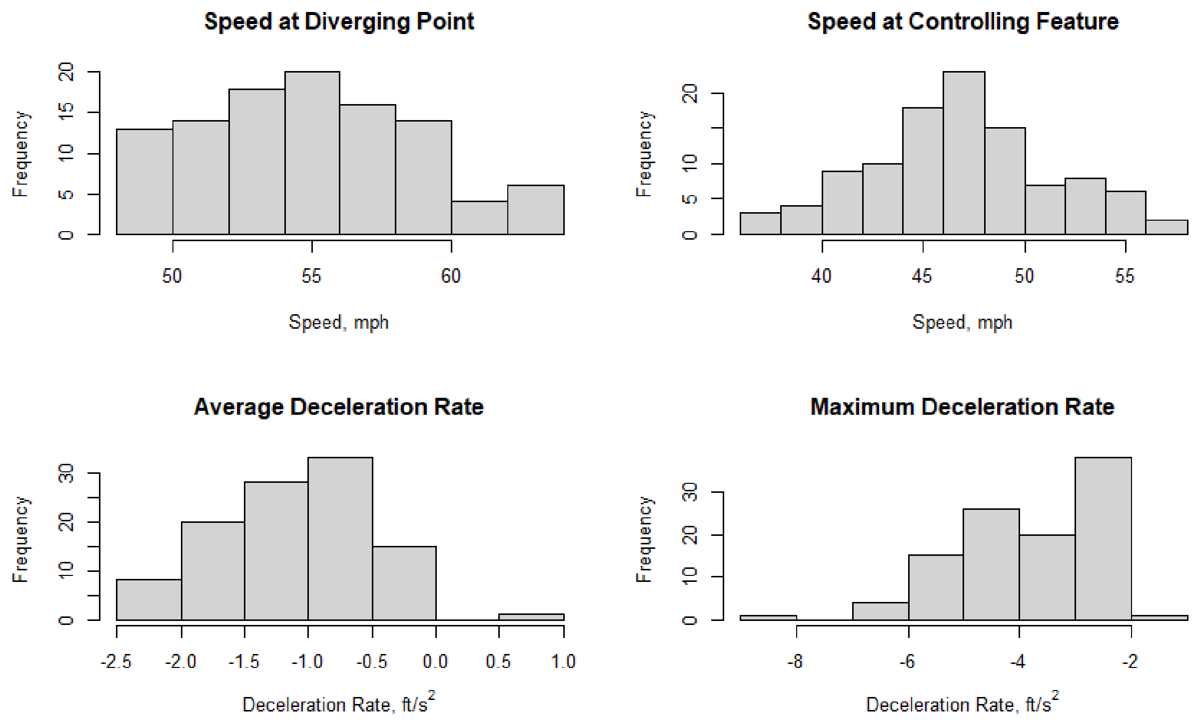


Figure 4-207. Distribution of Parameters for Site PA-11.

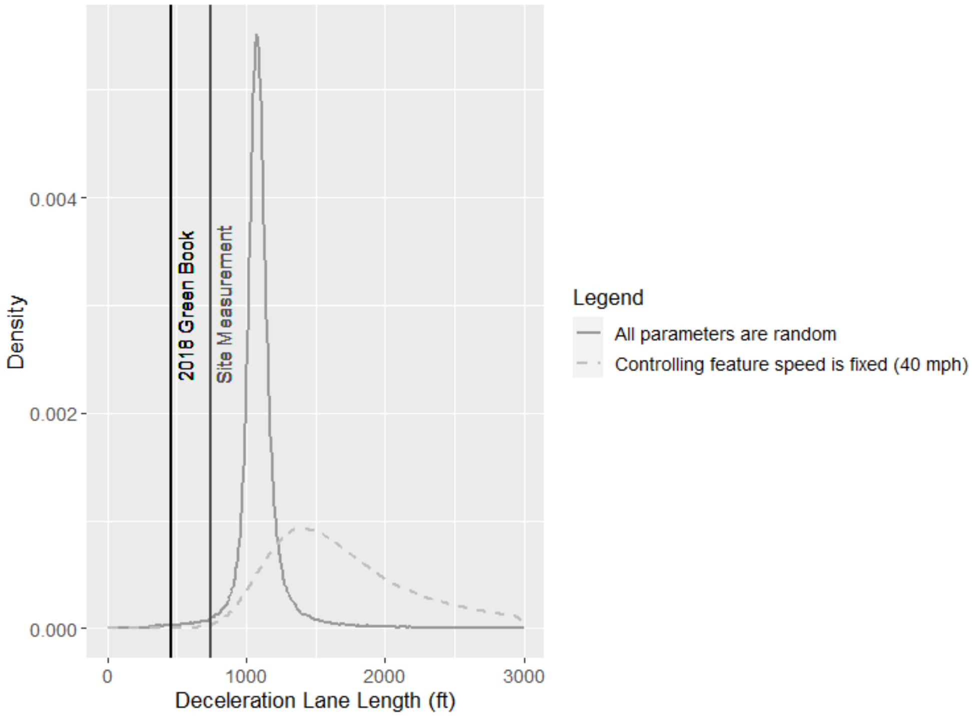


Figure 4-208. Frequency Distribution of Deceleration Lane Length based on Average Deceleration Rate for Site PA-11.

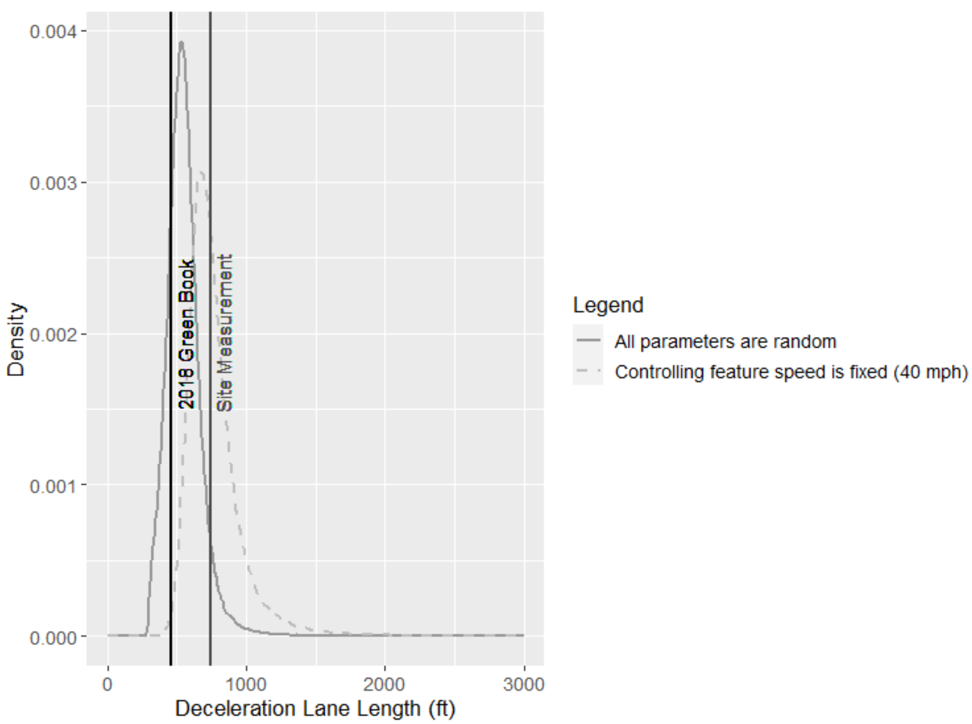


Figure 4-209. Frequency Distribution of Deceleration Lane Length based on Maximum Deceleration Rate for Site PA-11.

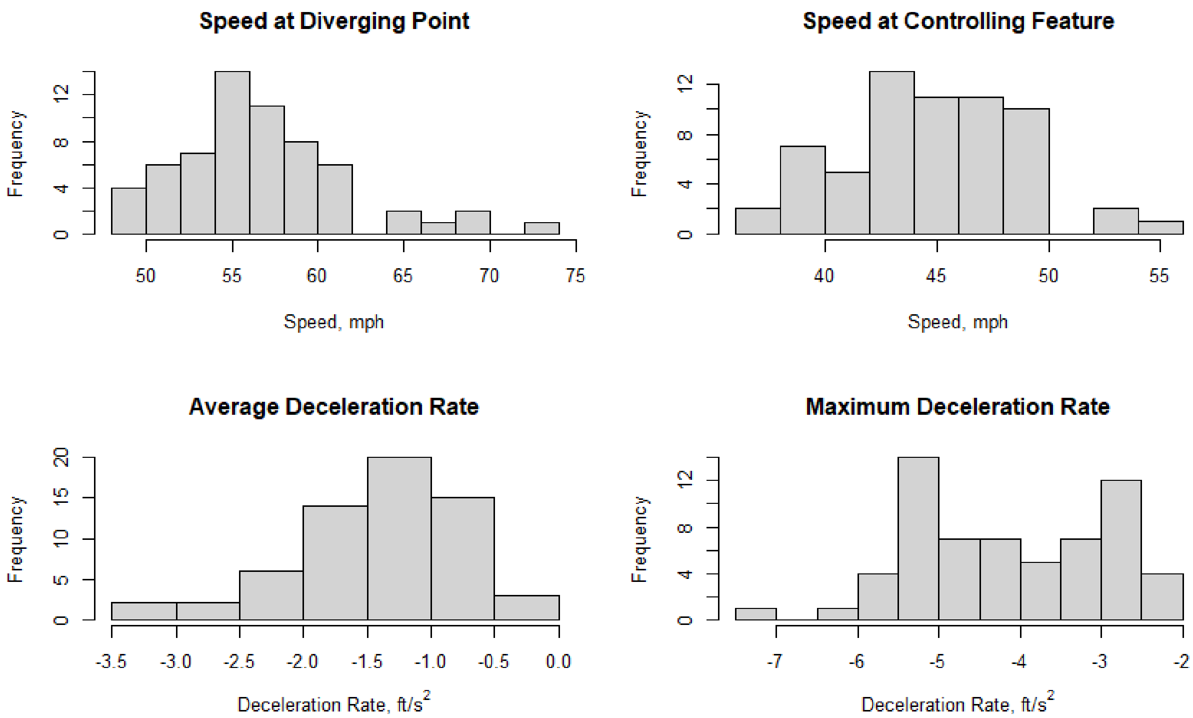


Figure 4-210. Distribution of Parameters for Site PA-12.

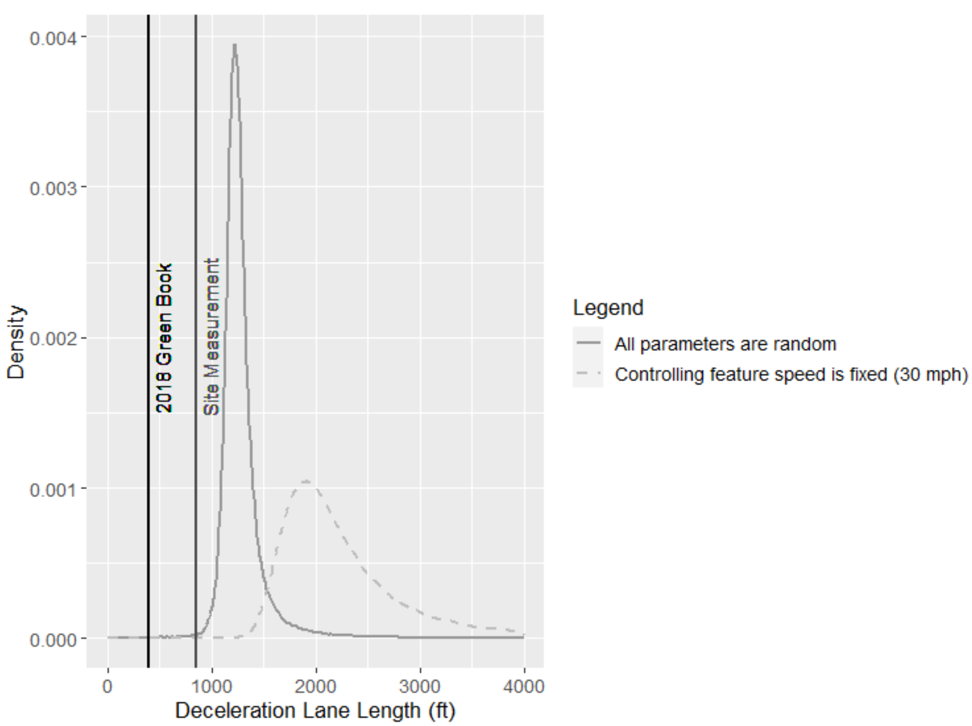


Figure 4-211. Frequency Distribution of Deceleration Lane Length based on Average Deceleration Rate for Site PA-12.

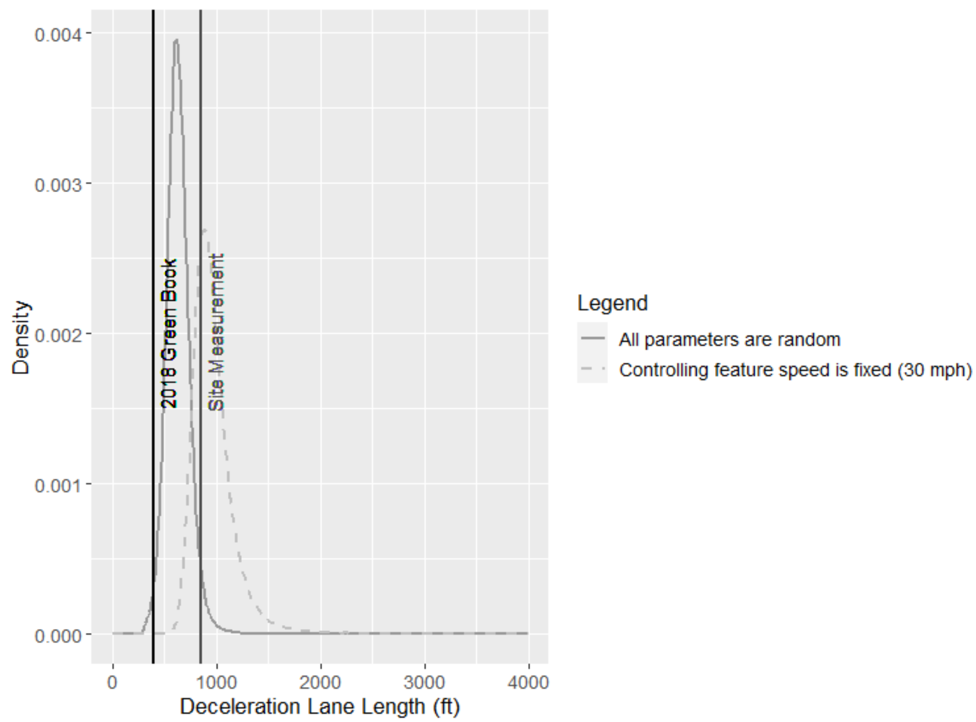


Figure 4-212. Frequency Distribution of Deceleration Lane Length based on Maximum Deceleration Rate for Site PA-12.

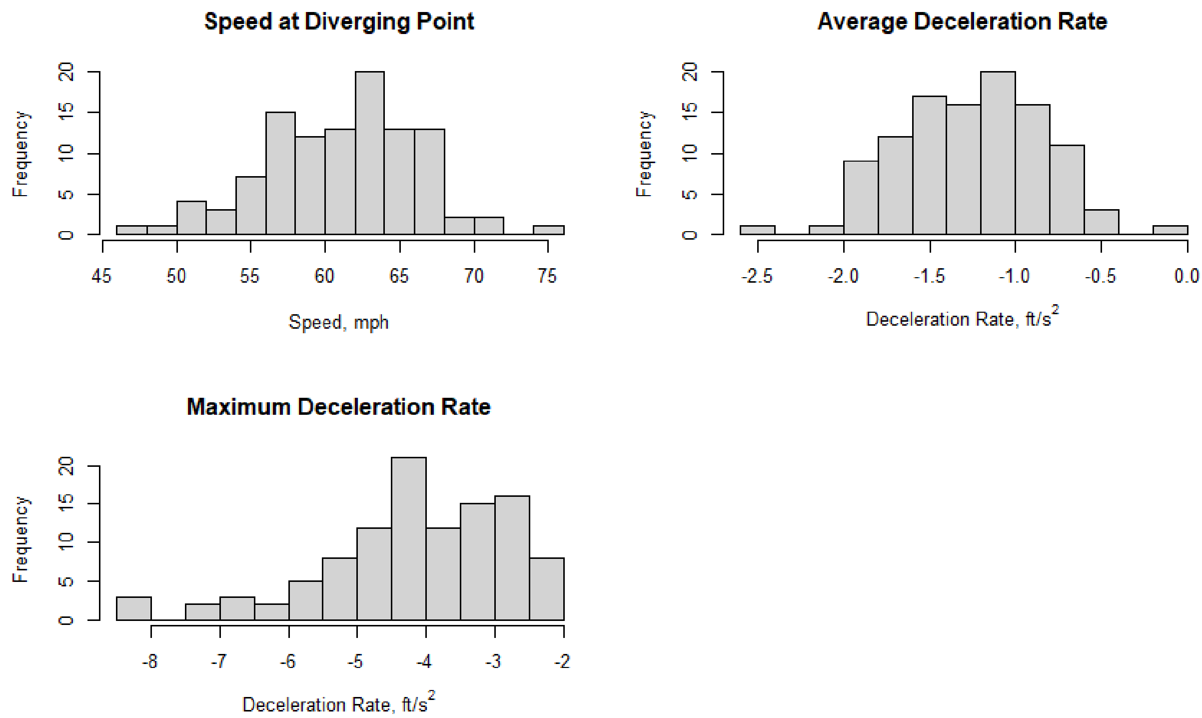


Figure 4-213. Distribution of Parameters for Site PA-13.

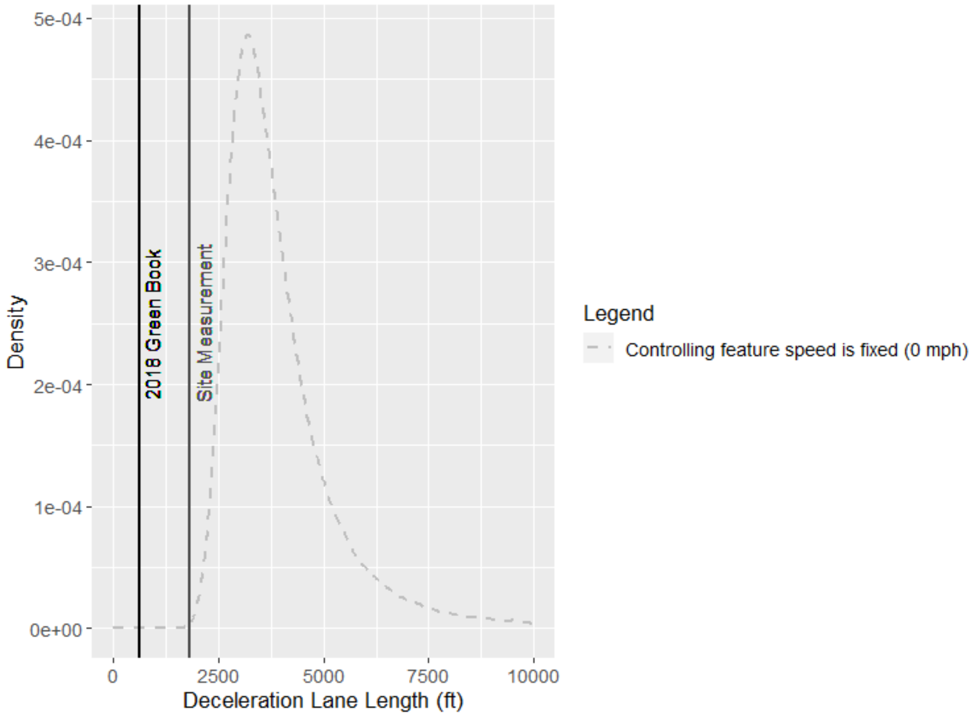


Figure 4-214. Frequency Distribution of Deceleration Lane Length based on Average Deceleration Rate for Site PA-13.

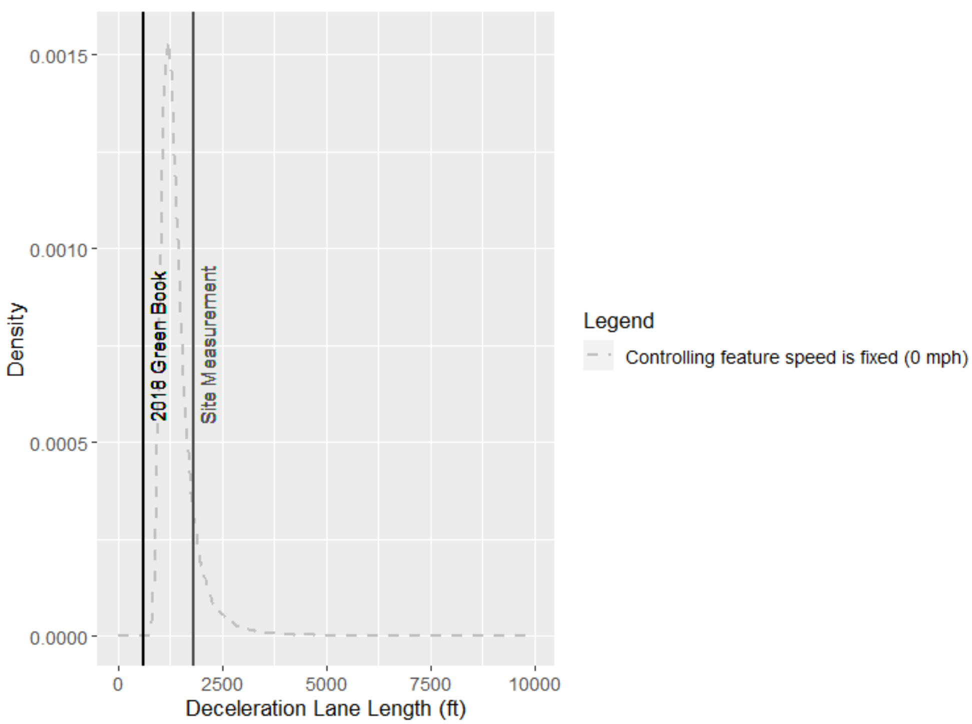


Figure 4-215. Frequency Distribution of Deceleration Lane Length based on Maximum Deceleration Rate for Site PA-13.

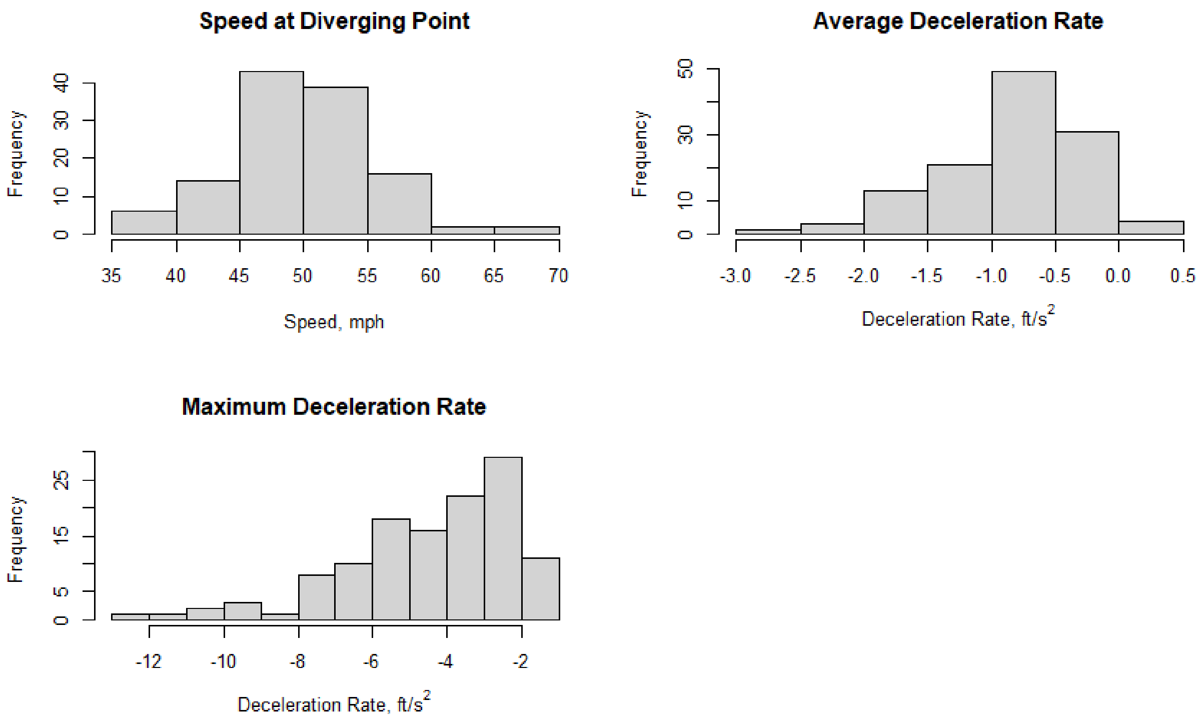


Figure 4-216. Distribution of Parameters for Site PA-14.

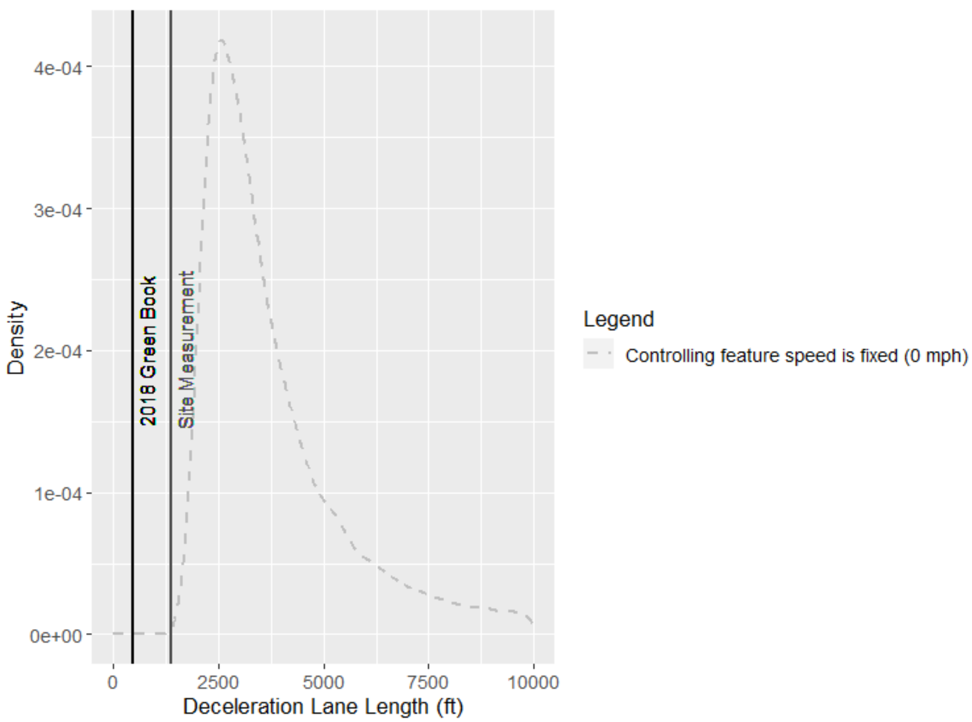


Figure 4-217. Frequency Distribution of Deceleration Lane Length based on Average Deceleration Rate for Site PA-14.

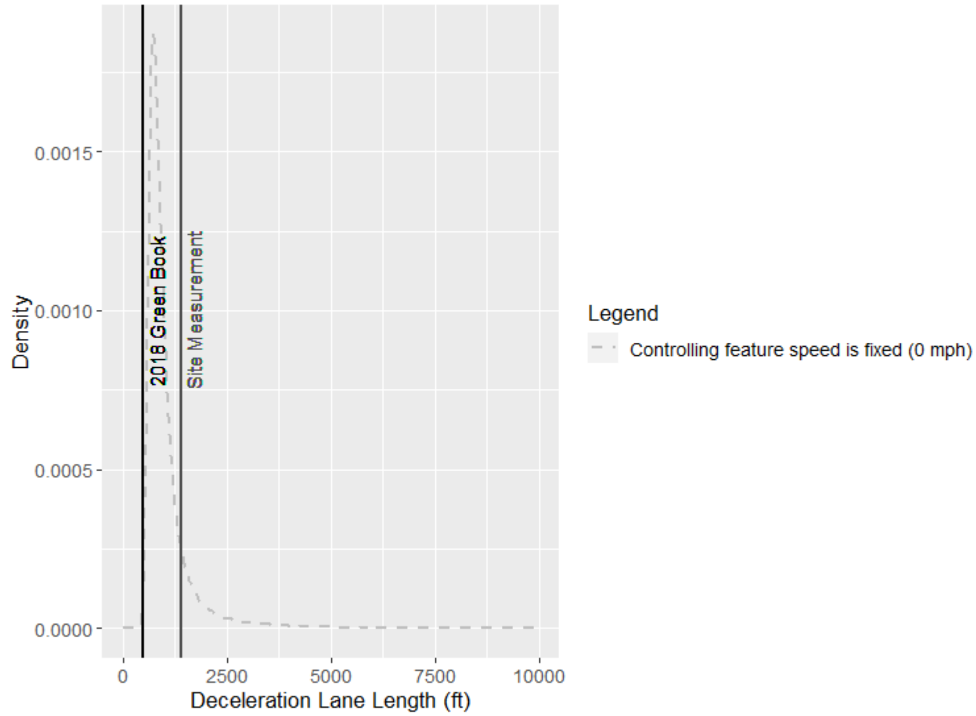


Figure 4-218. Frequency Distribution of Deceleration Lane Length based on Maximum Deceleration Rate for Site PA-14.

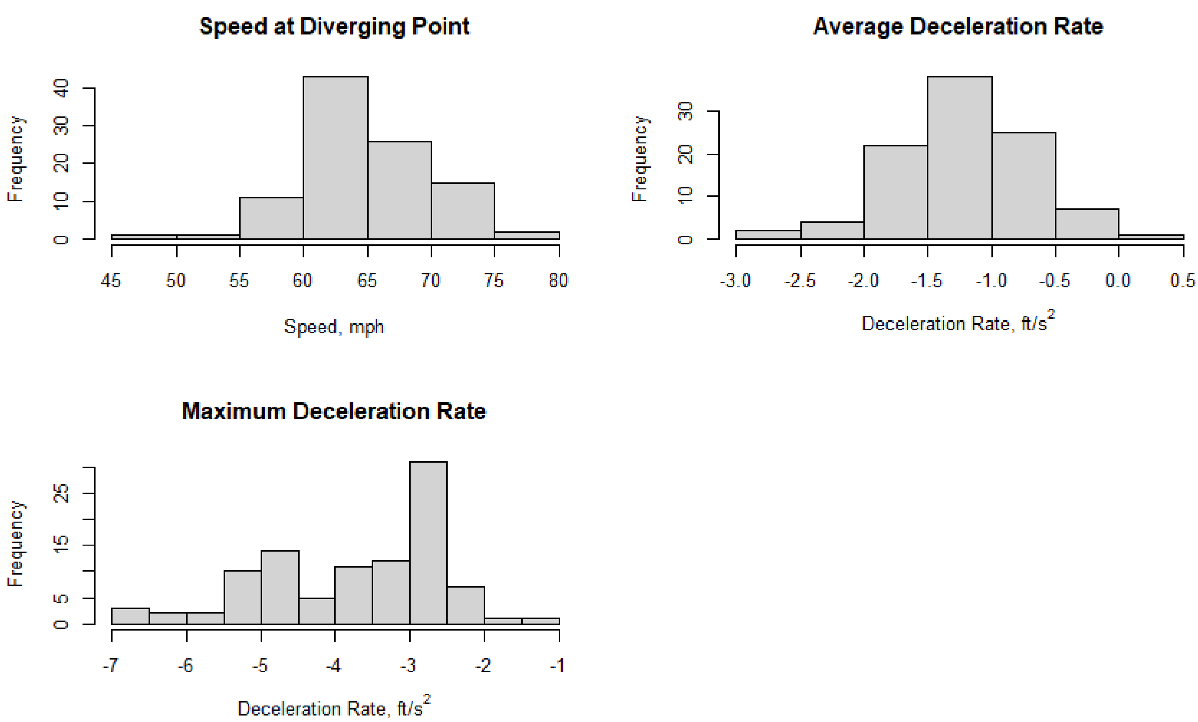


Figure 4-219. Distribution of Parameters for Site PA-15.

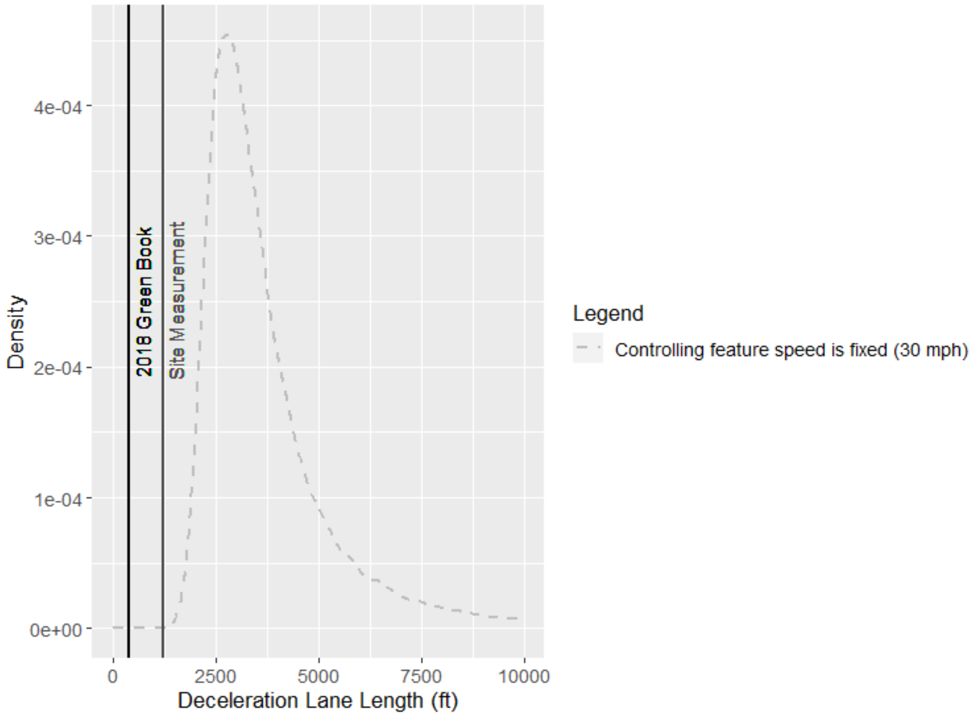


Figure 4-220. Frequency Distribution of Deceleration Lane Length based on Average Deceleration Rate for Site PA-15.

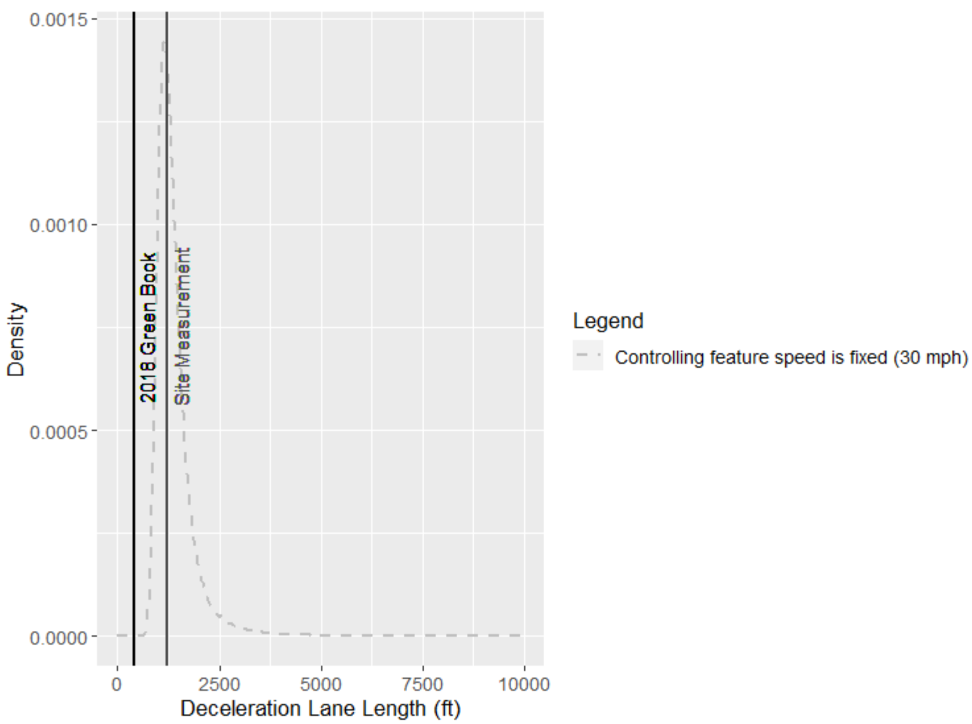


Figure 4-221. Frequency Distribution of Deceleration Lane Length based on Maximum Deceleration Rate for Site PA-15.

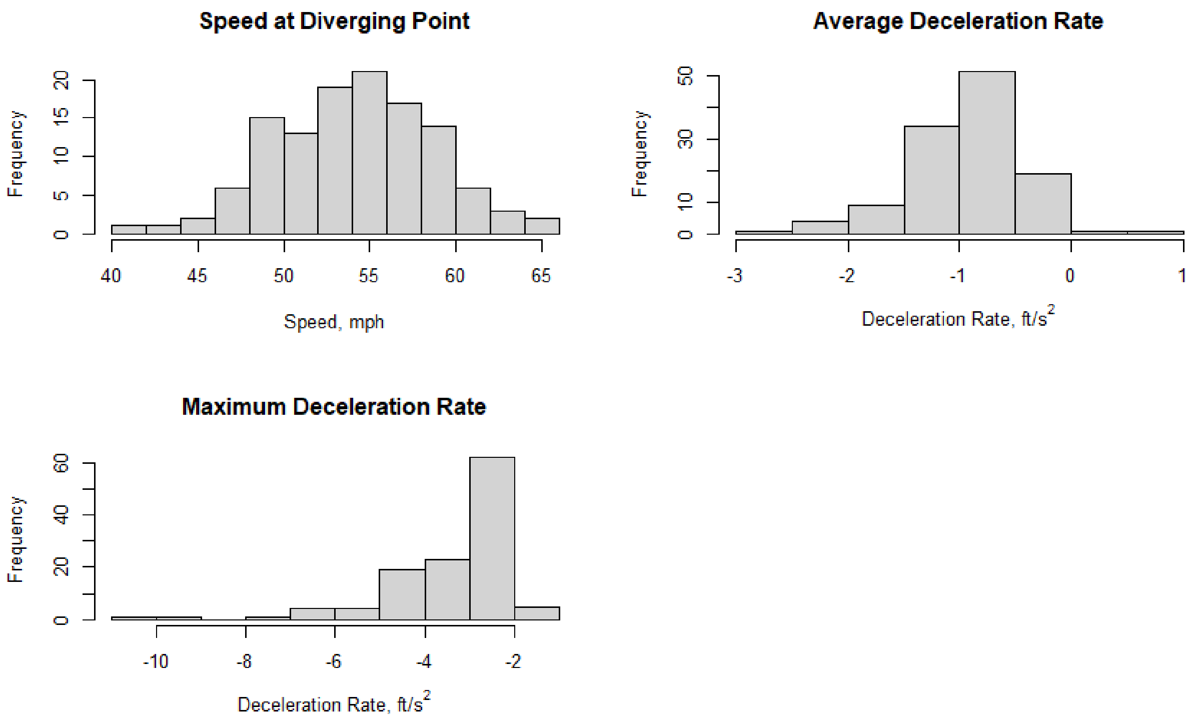


Figure 4-222. Distribution of Parameters for Site PA-16.

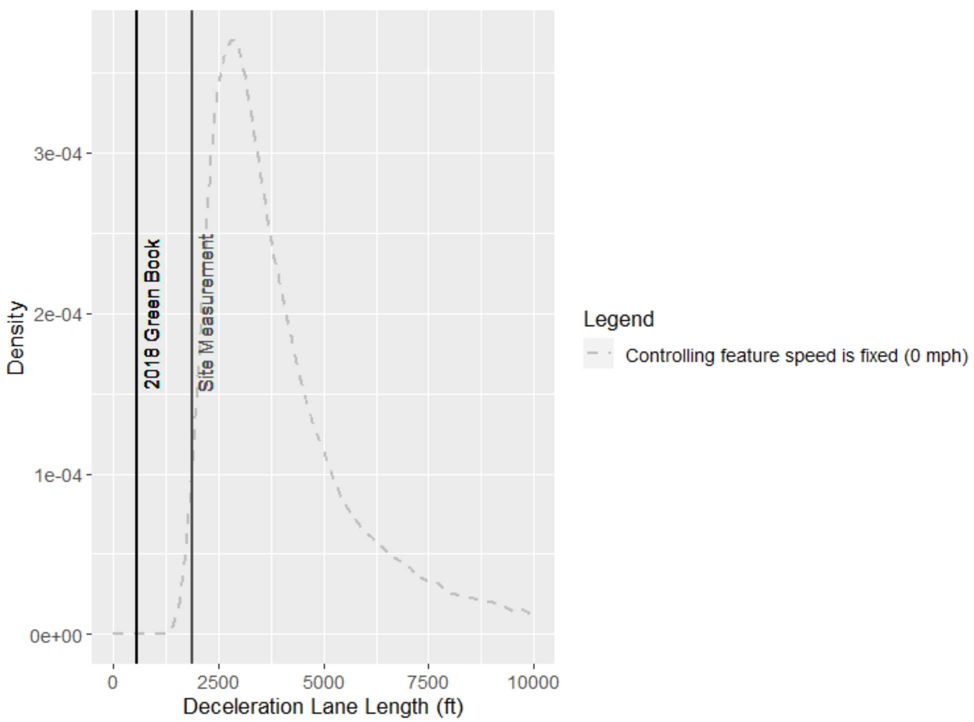


Figure 4-223. Frequency Distribution of Deceleration Lane Length based on Average Deceleration Rate for Site PA-16.

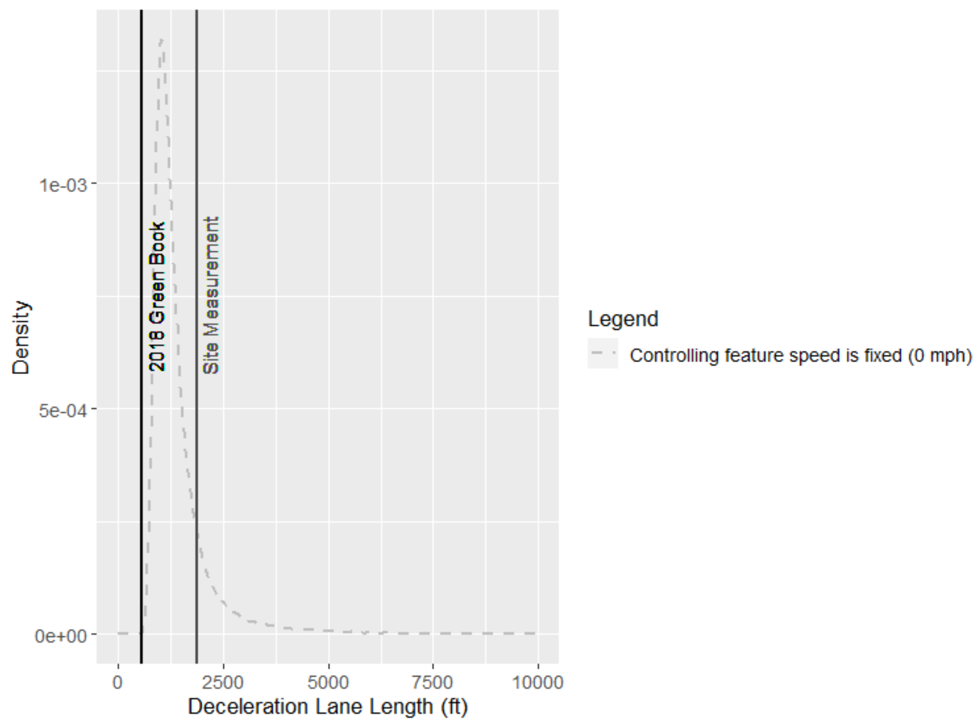


Figure 4-224. Frequency Distribution of Deceleration Lane Length based on Maximum Deceleration Rate for Site PA-16.

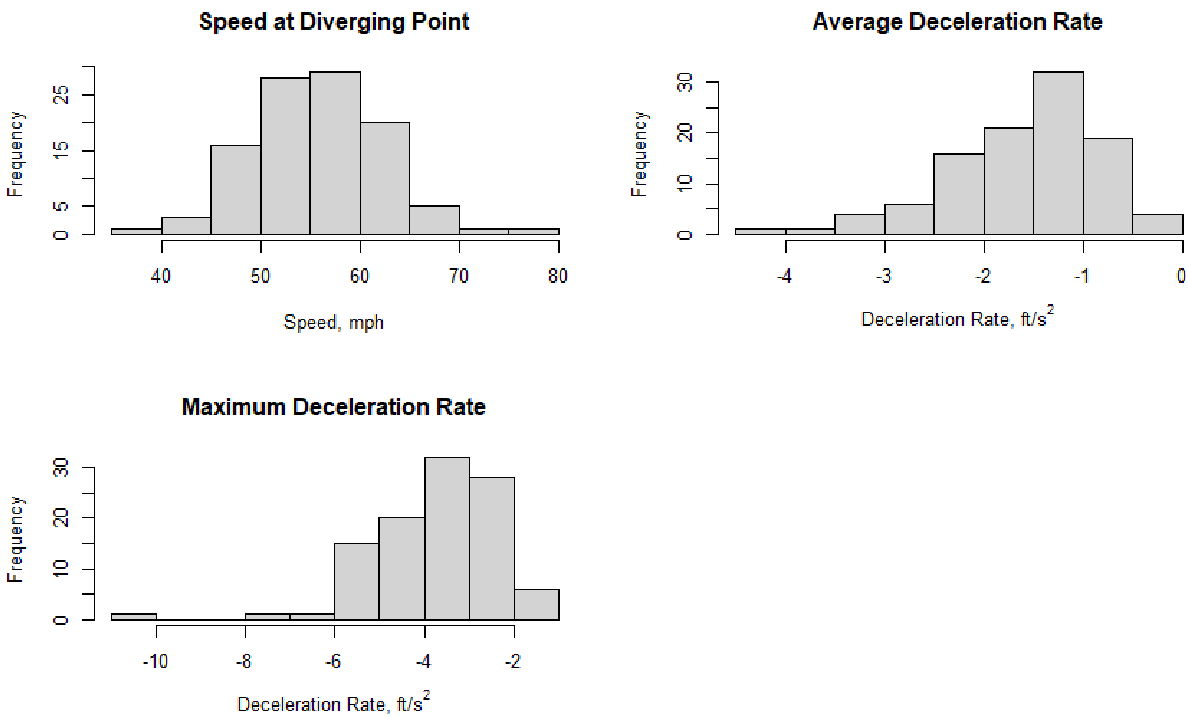


Figure 4-225. Distribution of Parameters for Site PA-17.

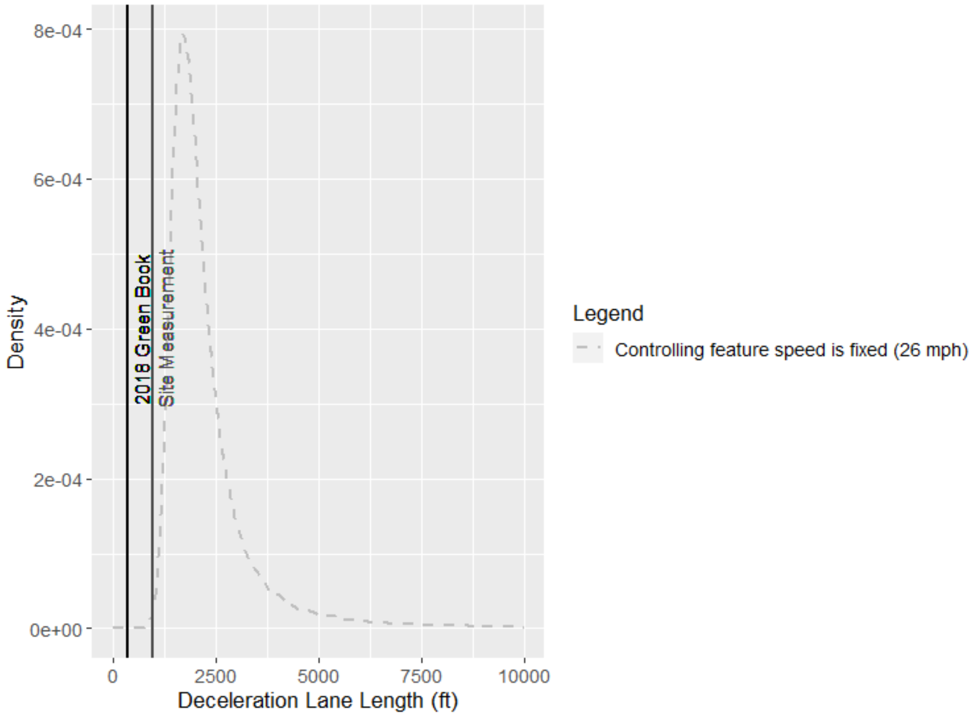


Figure 4-226. Frequency Distribution of Deceleration Lane Length based on Average Deceleration Rate for Site PA-17.

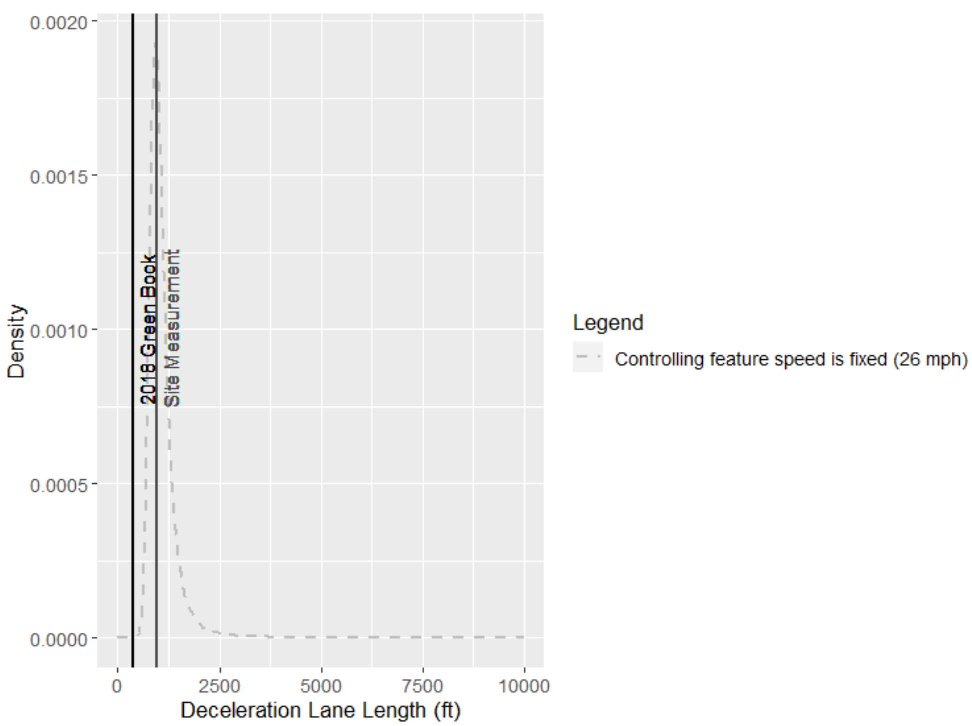


Figure 4-227. Frequency Distribution of Deceleration Lane Length based on Maximum Deceleration Rate for Site PA-1.

Appendix D Recommended Revisions to the 2018 AASHTO Green Book

This appendix presents the proposed revisions to the 2018 edition of the AASHTO publication, *A Policy on Geometric Design of Highways and Streets,* known as the Green Book. These revisions are the recommendations of the research team for NCHRP Project 15-75 and have not been approved by NCHRP or any AASHTO committee nor formally accepted for AASHTO publication.

Revisions are proposed to sections of the 2018 Green Book related to SSD criteria, acceleration lane design, and deceleration lane design. Generally speaking, revisions to the 2018 Green Book, including numerical values along with corresponding text, tables, figures, and/or equations, were proposed for either of the following cases:

- The 2018 Green Book guidance provided an insufficient design compared to the findings of this work, or
- The 2018 Green Book guidance provided an overly conservative design compared to the findings of this work.

Editorial revisions were also made to provide additional clarity and/or consistency between sections. No revisions were proposed for sections where the 2018 Green Book guidance was either in general alignment with the research findings or not sufficiently investigated in this work.

The numbering displayed for each section header represents the corresponding section number within the 2018 Green Book. Deletions are shown as ~~strikeouts~~ and additions are shown in red.

3.2 Sight Distance

3.2.2 Stopping Sight Distance

3.2.2.1 Brake Reaction Time

Brake reaction time is the interval from the instant that the driver recognizes the existence of an obstacle on the roadway ahead that necessitates braking until the instant that the driver actually applies the brakes. Under certain conditions, such as emergency situations denoted by flares or flashing lights, drivers accomplish these tasks almost instantly. Under most other conditions, the driver needs not only to see the object but also to recognize it as a stationary or slowly moving object against the background of the roadway and other objects, such as walls, fences, trees, poles, or bridges. Such determinations take time, and the amount of time needed varies considerably with the distance to the object, the visual acuity of the driver, the driver's reaction time, the atmospheric visibility, the type and the condition of the roadway, and the nature of the obstacle. Vehicle speed and roadway environment probably also influence reaction time. Normally, a driver traveling at or near the design speed is more alert than one traveling at a lesser speed. A driver on a street in an urban area confronted by innumerable potential conflicts with parked vehicles, driveways, and cross streets is also likely to be more alert than the same driver on a limited-access facility where such conditions should be almost nonexistent. However, a driver on an urban street faces a high mental workload in trying to monitor additional conflicts, so there is no assurance that the driver will be able to quickly detect a need for immediate action from among the many potential sources of conflict.

The study of reaction times by Johansson and Rumar *(41)* referred to in Section 2.2.6 was based on data from 321 drivers who expected to apply their brakes. The median reaction-time value for these drivers was 0.66 s, with 10 percent using 1.5 s or longer. These findings correlate with those of earlier studies in which alerted drivers were also evaluated. Another study *(46)* found 0.64 s as the average reaction time, while 5 percent of the drivers needed over 1 s. In a third study *(50)*, the values of brake reaction time ranged from 0.4 to 1.7 s. In the Johansson and Rumar study *(41)*, when the event that prompted application of the brakes was unexpected, drivers' response times were found to increase by approximately 1 s or more; some reaction times were greater than 1.5 s. This increase in reaction time substantiated earlier laboratory and road tests in which the conclusion was drawn that a driver who needed 0.2 to 0.3 s of reaction time under alerted conditions would need 1.5 s of reaction time under normal conditions. Minimum brake reaction times for drivers could thus be at least 1.64 s, 0.64 s for alerted drivers plus 1 s for the unexpected event. Because the studies discussed above used simple prearranged signals, they represent the least complex of roadway conditions. Even under these simple conditions, it was found that some drivers took over 3.5 s to respond.

Because actual conditions on the highway are generally more complex than those of the studies, ~~and because there is wide variation in driver reaction times, it is evident that the criterion adopted for use should be greater than 1.64 s.~~ NCHRP 15-75 analyzed brake response data from a sample of 4,735 crash or near-crash events across various roadway contexts that were collected as a part of the second Strategic Highway Research Program (SHRP2) Naturalistic Driving Study (NDS) (*NCHRP 15-75*). Several subsets of the data were also investigated, including by roadway context (rural, rural town/suburban, and urban/urban core), initial speed (low, medium, high), secondary task involvement, and animal involvement. The overall average and 90th- percentile reaction times from the NDS crash/near-crash events were approximately 1.3 and 2.2-s, respectively, with little variation observed across the aforementioned categories. The only exception was for cases where an animal was involved (2.5 percent of all events), which reduced the average brake response time to 0.65 s. The average reaction time for these crash/near-crash events with no secondary tasks involved (1.12 s) was very similar to the average reaction time observed (1.14 s) in NCHRP 400 (*19*).

The brake reaction time used in design should be long enough to include the reaction times needed by nearly all drivers under most highway conditions. NCHRP 15-75 found that 10 percent of drivers involved in a crash or near-crash event utilized a brake reaction time greater than 2.2 s (*NCHRP 15-75*). ~~Studies documented in the literature show that a 2.5-s brake reaction time for stopping sight situations encompasses~~

~~the capabilities of most drivers, including those of older drivers.~~ Although the recommended design criterion of 2.2-s ~~2.5-s~~ for brake reaction time is slightly faster than the 2.5-s utilized in prior versions of the AASHTO Green Book, it represents ~~exceeds~~ the 90th percentile of reaction time for ~~all~~ drivers involved in a crash or near-crash event and was used in the development of Table 3-1. A brake reaction time of 2.2-s ~~2.5-s~~ is considered adequate for conditions that are more complex than the simple conditions used in laboratory and road tests, but it may not be ~~is not~~ adequate for the most complex conditions encountered in actual driving. The need for greater reaction time in the most complex conditions encountered on the roadway, such as those found at multiphase at-grade intersections and at ramp terminals on through roadways, can be found in Section 3.2.3, "Decision Sight Distance."

3.2.2.2 Braking Distance

The approximate braking distance of a vehicle on a level roadway traveling at the design speed of the roadway may be determined from the following:

U. S. Customary	Metric	
$d_B = 1.075\frac{V^2}{a}$ where: d_B = braking distance, ft V = design speed, mph a = deceleration rate, ft/s^2	$d_B = 0.039\frac{V^2}{a}$ where: d_B = braking distance, m V = design speed, km/h a = deceleration rate, m/s^2	(3-1)

~~Studies documented in the literature show~~ NCHRP 400 *(19)* found that most drivers decelerate at a rate greater than 14.8 ft/s^2 [4.5 m/s^2] when confronted with the need to stop for an unexpected object in the roadway. NCHRP 400 also found that approximately 90 percent of all drivers decelerate at rates greater than 11.2 ft/s^2 [3.4 m/s^2]. More recently, deceleration data were analyzed from a sample of 4,735 crash or near-crash events across various roadway contexts and speeds that were collected as a part of the SHRP2 NDS (*NCHRP 15-75*). The maximum deceleration rate utilized during each crash or near-crash event was analyzed, as this value represents driver braking capabilities. The maximum deceleration rates were found to be lower in higher speed contexts, such as rural areas, where the 10th-percentile and average deceleration rates were 11.8 ft/s^2 and 20.4 ft/s^2, respectively. The maximum deceleration rates were higher in lower-speed contexts, such as urban areas (including urban core), where the 10th-percentile and average deceleration rates were 15.0 ft/s^2 and 22.8 ft/s^2, respectively. Deceleration rates were also found to be lower if no secondary task was involved, but higher if an animal was involved.

Such decelerations are within the driver's capability to stay within his or her lane and maintain steering control during the braking maneuver on wet surfaces. Therefore, 11.8 ~~11.2~~ ft/s^2 [3.6 ~~3.4~~ m/s^2] ~~(a comfortable deceleration for most drivers)~~ is recommended as the default deceleration threshold for determining stopping sight distance, particularly in rural areas and on all high-speed roadways (greater than 45 mph). However, on lower speed (less than or equal to 45 mph) streets in urban areas, a deceleration rate of 15.0 ft/s^2 [4.5 m/s^2] may be utilized for determining stopping sight distance. Implicit in the choice of this deceleration threshold is the assessment that most vehicle braking systems and the tire-pavement friction levels of most roadways are capable of providing a deceleration rate of at least 15.0 ~~11.2~~ ft/s^2 [4.5 ~~3.4~~ m/s^2]. The friction available on most wet pavement surfaces and the capabilities of most vehicle braking systems can provide braking friction that exceeds this deceleration rate.

~~***Table 3-1. Stopping Sight Distance on Level Roadways***~~

~~U.S. Customary~~				~~Metric~~			
			~~Stopping Sight Distance~~				~~Stopping Sight Distance~~

~~Design Speed (mph)~~	~~Brake Reaction Distance (ft)~~	~~Braking Distance on Level (ft)~~	~~Calculated (ft)~~	~~Design (ft)~~
~~15~~	~~55.1~~	~~21.6~~	~~76.7~~	~~80~~
~~20~~	~~73.5~~	~~38.4~~	~~111.9~~	~~115~~
~~25~~	~~91.9~~	~~60.0~~	~~151.9~~	~~155~~
~~30~~	~~110.3~~	~~86.4~~	~~196.7~~	~~200~~
~~35~~	~~128.6~~	~~117.6~~	~~246.2~~	~~250~~
~~40~~	~~147.0~~	~~153.6~~	~~300.6~~	~~305~~
~~45~~	~~165.4~~	~~194.4~~	~~359.8~~	~~360~~
~~50~~	~~183.8~~	~~240.0~~	~~423.8~~	~~425~~
~~55~~	~~202.1~~	~~290.3~~	~~492.4~~	~~495~~
~~60~~	~~220.5~~	~~345.5~~	~~566.0~~	~~570~~
~~65~~	~~238.9~~	~~405.5~~	~~644.4~~	~~645~~
~~70~~	~~257.3~~	~~470.3~~	~~727.6~~	~~730~~
~~75~~	~~275.6~~	~~539.9~~	~~815.5~~	~~820~~
~~80~~	~~294.0~~	~~614.3~~	~~908.3~~	~~910~~
~~85~~	~~313.5~~	~~693.5~~	~~1007.0~~	~~1010~~

~~Design Speed (km/h)~~	~~Brake Reaction Distance (m)~~	~~Braking Distance on Level (m)~~	~~Calculated (m)~~	~~Design (m)~~
~~20~~	~~13.9~~	~~4.6~~	~~18.5~~	~~20~~
~~30~~	~~20.9~~	~~10.3~~	~~31.2~~	~~35~~
~~40~~	~~27.8~~	~~18.4~~	~~46.2~~	~~50~~
~~50~~	~~34.8~~	~~28.7~~	~~63.5~~	~~65~~
~~60~~	~~41.7~~	~~41.3~~	~~83.0~~	~~85~~
~~70~~	~~48.7~~	~~56.2~~	~~104.9~~	~~105~~
~~80~~	~~55.6~~	~~73.4~~	~~129.0~~	~~130~~
~~90~~	~~62.6~~	~~92.9~~	~~155.5~~	~~160~~
~~100~~	~~69.5~~	~~114.7~~	~~184.2~~	~~185~~
~~110~~	~~76.5~~	~~138.8~~	~~215.3~~	~~220~~
~~120~~	~~83.4~~	~~165.2~~	~~248.6~~	~~250~~
~~130~~	~~90.4~~	~~193.8~~	~~284.2~~	~~285~~
~~140~~	~~97.3~~	~~224.8~~	~~322.1~~	~~325~~

Source: *A Policy on Geometric Design of Highways and Streets* (2018) by AASHTO, Washington, D.C. Used with permission.

Proposed Table 3-1. Stopping Sight Distance on Level Roadways

RURAL OR HIGH SPEED

U.S. Customary				
Design Speed (mph)	Brake Reaction Distance (ft)	Braking Distance on Level (ft)	Stopping Sight Distance	
			Calculated (ft)	Design (ft)
15	48.5	20.5	69.0	70
20	64.7	36.4	101.1	105
25	80.9	56.9	137.8	140
30	97.0	82.0	179.0	180
35	113.2	111.6	224.8	225
40	129.4	145.8	275.1	280
45	145.5	184.5	330.0	335
50	161.7	227.8	389.5	390
55	177.9	275.6	453.5	455
60	194.0	328.0	522.0	525
65	210.2	384.9	595.1	600
70	226.4	446.4	672.8	675
75	242.6	512.4	755.0	760
80	258.7	583.1	841.8	845
85	274.9	658.2	933.1	935

Metric				
Design Speed (km/h)	Brake Reaction Distance (m)	Braking Distance on Level (m)	Stopping Sight Distance	
			Calculated (m)	Design (m)
20	12.2	4.3	16.6	20
30	18.3	9.8	28.1	30
40	24.5	17.3	41.8	45
50	30.6	27.1	57.7	60
60	36.7	39.0	75.7	80
70	42.8	53.1	95.9	100
80	48.9	69.3	118.3	120
90	55.0	87.8	142.8	145
100	61.2	108.3	169.5	170
110	67.3	131.1	198.4	200
120	73.4	156.0	229.4	230
130	79.5	183.1	262.6	265
140	85.6	212.3	298.0	300

LOW SPEED URBAN

U.S. Customary				
Design Speed (mph)	Brake Reaction Distance (ft)	Braking Distance on Level (ft)	Stopping Sight Distance	
			Calculated (ft)	Design (ft)
15	48.5	16.1	64.6	65
20	64.7	28.7	93.3	95
25	80.9	44.8	125.6	130
30	97.0	64.5	161.5	165
35	113.2	87.8	201.0	205
40	129.4	114.7	244.0	245
45	145.5	145.1	290.7	295

Metric				
Design Speed (km/h)	Brake Reaction Distance (m)	Braking Distance on Level (m)	Stopping Sight Distance	
			Calculated (m)	Design (m)
20	12.2	3.5	15.7	20
30	18.3	7.8	26.1	30
40	24.5	13.9	38.3	40
50	30.6	21.7	52.2	55
60	36.7	31.2	67.9	70
70	42.8	42.5	85.3	90

3.2.2.3 Design Values

The stopping sight distance is the sum of the distance traversed during the brake reaction time and the distance to brake the vehicle to a stop. The computed distances for various speeds at the assumed conditions on level roadways are shown in Table 3-1 and were developed from the following equation:

U. S. Customary	Metric	
$SSD = 1.47Vt + 1.075\frac{V^2}{a}$	$SSD = 0.278Vt + 0.039\frac{V^2}{a}$	(3-2)
where:	where:	
SSD = stopping sight distance, ft	SSD = stopping sight distance, m	
V = design speed, mph	V = design speed, km/h	
t = brake reaction time, 2.2 ~~2.5~~ s	t = brake reaction time, 2.2 ~~2.5~~ s	
a = deceleration rate, ft/s^2	a = deceleration rate, m/s^2	

3.2.2.4 Effect of Grade on Stopping

When a highway is on a grade, Equation 3-1 for braking distance is modified as follows:

U. S. Customary	Metric	
$d_B = \frac{V^2}{30\left[\left(\frac{a}{32.2} \pm G\right)\right]}$	$d_B = \frac{V^2}{254\left[\left(\frac{a}{9.81} \pm G\right)\right]}$	(3-3)
where:	where:	
d_B = braking distance on grade, ft	d_B = braking distance, m	
V = design speed, mph	V = design speed, km/h	
a = deceleration, ft/s^2	a = deceleration, m/s^2	
G = grade, rise/run, ft/ft	G = grade, rise/run, m/m	

In this equation, G is the rise in elevation divided by the distance of the run and the percent of grade divided by 100, and the other terms are as previously stated. The stopping distances needed on upgrades are shorter than on level roadways; those on downgrades are longer. The stopping sight distances for various grades shown in Table 3-2 are the values determined by using Equation 3-3 in place of the second term in Equation 3-2. These adjusted sight distance values are computed ~~for wet-pavement conditions~~ using the same design speeds, brake reaction times, and deceleration rates used for level roadways in Table 3-1.

~~Table 3-2 Stopping Sight Distance on Grades~~

~~U.S. Customary~~

~~Design Speed (mph)~~	~~Stopping Sight Distance (ft)~~					
	~~Downgrades~~			~~Upgrades~~		
	~~3%~~	~~6%~~	~~9%~~	~~3%~~	~~6%~~	~~9%~~
~~15~~	~~80~~	~~82~~	~~85~~	~~75~~	~~74~~	~~73~~
~~20~~	~~116~~	~~120~~	~~126~~	~~109~~	~~107~~	~~104~~
~~25~~	~~158~~	~~165~~	~~173~~	~~147~~	~~143~~	~~140~~
~~30~~	~~205~~	~~215~~	~~227~~	~~200~~	~~184~~	~~179~~
~~35~~	~~257~~	~~271~~	~~287~~	~~237~~	~~229~~	~~222~~
~~40~~	~~315~~	~~333~~	~~354~~	~~289~~	~~278~~	~~269~~
~~45~~	~~378~~	~~400~~	~~427~~	~~344~~	~~331~~	~~320~~
~~50~~	~~446~~	~~474~~	~~507~~	~~405~~	~~388~~	~~375~~
~~55~~	~~520~~	~~553~~	~~593~~	~~469~~	~~450~~	~~433~~
~~60~~	~~598~~	~~638~~	~~686~~	~~538~~	~~515~~	~~495~~
~~65~~	~~682~~	~~728~~	~~785~~	~~612~~	~~584~~	~~561~~
~~70~~	~~771~~	~~825~~	~~891~~	~~690~~	~~658~~	~~631~~
~~75~~	~~866~~	~~927~~	~~1003~~	~~772~~	~~736~~	~~704~~
~~80~~	~~965~~	~~1035~~	~~1121~~	~~859~~	~~817~~	~~782~~
~~85~~	~~1070~~	~~1149~~	~~1246~~	~~949~~	~~902~~	~~862~~

~~Metric~~

~~Design Speed (km/h)~~	~~Stopping Sight Distance (m)~~					
	~~Downgrades~~			~~Upgrades~~		
	~~3%~~	~~6%~~	~~9%~~	~~3%~~	~~6%~~	~~9%~~
~~20~~	~~20~~	~~20~~	~~20~~	~~19~~	~~18~~	~~18~~
~~30~~	~~32~~	~~35~~	~~35~~	~~31~~	~~30~~	~~29~~
~~40~~	~~50~~	~~50~~	~~53~~	~~45~~	~~44~~	~~43~~
~~50~~	~~66~~	~~70~~	~~74~~	~~61~~	~~59~~	~~58~~
~~60~~	~~87~~	~~92~~	~~97~~	~~80~~	~~77~~	~~75~~
~~70~~	~~110~~	~~116~~	~~124~~	~~100~~	~~97~~	~~93~~
~~80~~	~~136~~	~~144~~	~~154~~	~~123~~	~~118~~	~~114~~
~~90~~	~~164~~	~~174~~	~~187~~	~~148~~	~~141~~	~~136~~
~~100~~	~~194~~	~~207~~	~~223~~	~~174~~	~~167~~	~~160~~
~~110~~	~~227~~	~~243~~	~~262~~	~~203~~	~~194~~	~~186~~
~~120~~	~~263~~	~~281~~	~~304~~	~~234~~	~~223~~	~~214~~
~~130~~	~~302~~	~~323~~	~~350~~	~~267~~	~~254~~	~~243~~
~~140~~	~~341~~	~~367~~	~~398~~	~~302~~	~~287~~	~~274~~

Source: *A Policy on Geometric Design of Highways and Streets* (2018) by AASHTO, Washington, D.C. Used with permission.

Proposed Table 3-2 Stopping Sight Distance on Grades
RURAL OR HIGH SPEED

U.S. Customary						
Design Speed (mph)	Stopping Sight Distance (ft)					
	Downgrades			Upgrades		
	3%	6%	9%	3%	6%	9%
15	71	73	76	68	67	65
20	105	109	113	99	96	94
25	143	149	157	134	130	127
30	187	195	206	173	168	163
35	235	247	261	217	209	203
40	288	304	323	264	255	247
45	347	366	390	316	304	294
50	410	434	464	372	358	345
55	478	507	543	433	415	399
60	551	586	629	497	476	457
65	629	670	720	566	541	519
70	712	760	818	639	610	585
75	800	855	921	716	683	654
80	893	955	1031	797	759	727
85	991	1061	1147	883	840	803

Metric						
Design Speed (km/h)	Stopping Sight Distance (m)					
	Downgrades			Upgrades		
	3%	6%	9%	3%	6%	9%
20	17	18	18	17	16	16
30	29	30	32	28	27	27
40	44	45	48	41	40	39
50	60	63	67	56	54	53
60	79	83	88	73	70	68
70	101	106	113	92	88	86
80	124	132	140	113	108	105
90	150	159	171	136	130	125
100	178	190	204	161	154	148
110	209	223	240	188	179	172
120	242	259	279	217	207	198
130	277	297	320	248	236	226
140	315	337	365	281	267	255

LOW SPEED URBAN

U.S. Customary						
Design Speed (mph)	Stopping Sight Distance (ft)					
	Downgrades			Upgrades		
	3%	6%	9%	3%	6%	9%
15	66	67	69	64	63	63
20	96	98	101	92	91	89
25	129	133	137	123	121	119
30	166	171	177	158	155	151
35	207	214	222	196	191	187
40	252	261	272	237	231	226
45	301	312	326	282	274	267

Metric						
Design Speed (km/h)	Stopping Sight Distance (m)					
	Downgrades			Upgrades		
	3%	6%	9%	3%	6%	9%
20	16	17	17	16	16	16
30	27	28	28	26	26	25
40	40	41	42	38	37	36
50	54	56	58	51	50	49
60	70	73	76	66	65	63
70	88	92	96	83	81	78

On nearly all roads and streets, the grade is traversed by traffic in both directions of travel, but the sight distance at any point on the highway generally is different in each direction, particularly on straight roads in rolling terrain. As a general rule, the sight distance available on downgrades is larger than on upgrades, more or less automatically providing the appropriate corrections for grade. This may explain why some designers do not adjust stopping sight distance because of grade. Exceptions are one-way roadways or streets, as on divided highways with independent profiles. For these separate roadways, adjustments for grade may be needed.

3.2.2.5.1 New Construction vs. Projects on Existing Roads

The stopping sight distance criteria in Tables 3-1 and 3-2 are appropriate for use in new construction projects where no constraints are present, since stopping sight distances that meet these criteria can typically be readily implemented. Sight distance improvements for projects on existing roads are often very costly. Prior ~~Recent~~ research (*35*) has found little or no difference in crash experience between crest vertical curves that meet the stopping sight distance criteria in Tables 3-1 and 3-2 and those that do not, except where a design feature where drivers may need to change direction or speed is hidden from the driver's view. Recent research has shown that crash frequency increases as the amount of available SSD decreases on high-speed freeways and rural two-lane highways (*NCHRP 15-75*). Therefore, ~~in most cases,~~ design elements at which the stopping sight distance is less than shown in Tables 3-1 and 3-2 should be improved if justified through a performance-based analysis. ~~may be left in place. However,~~ This is especially true for cases where a roadway feature such as a horizontal curve, an intersection, a driveway, or a ramp terminal is hidden from the driver's view by the sight distance limitation or where a crash history review as part of the project development process finds a documented crash pattern that may be correctable by a sight distance improvement, improvement of stopping sight distance to the criteria presented in Tables 3-1 and 3-2 should be considered.

3.2.4 Passing Sight Distance for Two-Lane Highways

3.2.4.2 Design Values

The design values for passing sight distance are presented in Table 3-4. A comparison between Tables 3-1 and 3-4 shows that more sight distance is needed to accommodate passing maneuvers on a two-lane highway than to provide stopping sight distance.

Research has verified that the passing sight distance values in Table 3-4 are consistent with field observation of passing maneuvers (*35*). This research used two theoretical models for the sight distance needs of passing drivers; both models were based on the assumption that a passing driver will abort the passing maneuver and return to his or her normal lane behind the passed vehicle if a potentially conflicting vehicle comes into view before reaching a critical position in the passing maneuver beyond which the passing driver is committed to complete the maneuver. The Glennon model (*28*) assumes that the critical position occurs where the passing sight distance to complete the maneuver is equal to the sight distance needed to abort the maneuver. The Hassan et al. model (*37*) assumes that the critical position occurs where the passing sight distances to complete or abort the maneuver are equal or where the passing and passed vehicles are abreast, whichever occurs first.

Minimum passing sight distances for design of two-lane highways incorporate certain assumptions about driver behavior. Actual driver behavior in passing maneuvers varies widely. To accommodate these variations in driver behavior, the design criteria for passing sight distance should accommodate the behavior of a high percentage of drivers, rather than just the average driver. The assumptions made in applying the Glennon and Hassan et al. models (*28, 37*) are as follows:

1. The speeds of the passing and opposing vehicles are equal and represent the design speed of the highway.
2. The passed vehicle travels at uniform speed and speed difference between the passing and passed vehicles is 12 mph [19 km/h].
3. The passing vehicle has sufficient acceleration capability to reach the specified speed difference relative to the passed vehicle by the time it reaches the critical position, which generally occurs about 40 percent of the way through the passing maneuver.
4. The lengths of the passing and passed vehicles are 19 ft [5.8 m], as shown for the P design vehicle in Section 2.8.1.
5. The passing driver's perception-reaction time in deciding to abort passing a vehicle is 1 s.
6. If a passing maneuver is aborted, the passing vehicle will use a deceleration rate of 11.2 ft/s^2 [3.4 m/s^2]~~,~~ ~~the same deceleration rate used in stopping sight distance design criteria~~.
7. For a completed or aborted pass, the space headway between the passing and passed vehicles is 1 s.
8. The minimum clearance between the passing and opposing vehicles at the point at which the passing vehicle returns to its normal lane is 1 s.

The application of the passing sight distance models using these assumptions is presented in NCHRP Report 605 (35).

Passing sight distance for use in design should be based on a single passenger vehicle passing a single passenger vehicle. While there may be occasions to consider multiple passings, where two or more vehicles pass or are passed, it is not practical to assume such conditions in developing minimum design criteria. Research has shown that longer sight distances are often needed for passing maneuvers when the passed vehicle, the passing vehicle, or both are trucks (33). Longer sight distances occur in design, and such locations can accommodate an occasional multiple passing maneuver or a passing maneuver involving a truck.

3.2.6 Criteria for Measuring Sight Distance

3.2.6.1 Height of Driver's Eye

~~For all sight distance calculations for passenger vehicles,~~ Previous AASHTO guidance ~~the height of the~~ based sight distance measurements on an assumed driver's eye ~~is considered to be~~ height of 3.50 ft [1.08 m] above the road surface. ~~This value is based on a study (*19*) that found average vehicle heights have decreased to 4.25 ft [1.30 m] with a comparable decrease in average eye heights to 3.50 ft [1.08 m].~~ Recent measurements of driver eye height for passenger vehicles (car, SUV, van, pickup) found that 95 percent of driver eye heights exceed 3.65 ft [1.11 m], while 90 percent exceed 3.75 ft [1.14 m]. ~~Because of various factors that appear to place practical limits on further decreases in passenger car heights and the relatively small increases in the lengths of vertical curves that would result from further changes that do occur~~, Thus, 3.75 ft [1.14 m] ~~3.50 ft [1.08 m]~~ is considered to be the appropriate height of driver's eye for measuring both stopping and passing sight distances. For large trucks, the driver eye height typically ranges from 7.00 ~~3.50~~ to 8.15 ~~7.90~~ ft [2.13 ~~1.80~~ to 2.48 ~~2.40 m~~]. The recommended value of truck driver eye height for design is 7.60 ft [2.33 m] above the road surface (*19, NCHRP 15-75*).

3.2.6.2 Height of Object

For stopping sight distance and decision sight distance calculations, the height of object is considered to be 2.00 ft [0.60 m] above the road surface. For passing sight distance calculations, the height of object is considered to be 3.75 ft [1.14 m] ~~3.50 ft [1.08 m]~~ above the road surface.

Stopping sight distance object—The selection of a 2.00-ft [0.60-m] object height was based on research indicating that objects with heights less than 2.00 ft [0.60 m] are seldom involved in crashes (*19*). Therefore, it is considered that an object 2.00 ft [0.60 m] in height is representative of the smallest object that involves risk to drivers. An object height of 2.00 ft [0.60 m] is representative of the height of automobile headlights. and taillights on some shorter sedans and coupes. ~~Using object heights of less than 2.00 ft [0.60 m] for stopping sight distance calculations would result in longer crest vertical curves without a documented decrease in the frequency or severity of crashes (*19*). Object height of less than 2.00 ft [0.60 m] could substantially increase construction costs because additional excavation would be needed to provide the longer crest vertical curves. It is also doubtful that the driver's ability to perceive situations involving risk of collisions would be increased because recommended stopping sight distances for high-speed design are beyond most drivers' capabilities to detect objects less than 2.00 ft [0.60 m] in height (*19*).~~ Recent research has found automobile taillight heights to be considerably taller than those observed decades ago, with 15th percentile and average heights of 2.79 ft [0.85 m] and 2.97 ft [0.91 m], respectively. Although taillights are common visual targets in stopping sight distance situations, they are not the only objects that must be considered for design. Thus, additional investigation is necessary before recommendations can be made to increase the object height above 2.00 ft [0.60 m]. One exception is for cases of a sag vertical curve underpassing a structure, where a taller object height reduces the available sight distance. For such cases, an object height of 3.0 ft [0.9 m] is recommended, which represents the average automobile taillight height

(*NCHRP 15-75*). Further detail on sight distances for sag vertical curves at undercrossings is provided in Section 3.4.6.4.

Passing sight distance object—An object height of 3.75 ft [1.14 m] ~~3.50 ft [1.08 m]~~ is adopted for passing sight distance. This object height ~~is based on a vehicle height of 4.35 ft [1.33 m], which represents the 15th percentile of vehicle heights in the current passenger car population, less an allowance of 0.85 ft [0.25 m], which~~ represents a near-maximum value for the portion of the vehicle height that needs to be visible for another driver to recognize a vehicle as such (*35*). Passing sight distances calculated on this basis are also considered adequate for night conditions because headlight beams of an opposing vehicle generally can be seen from a greater distance than a vehicle can be recognized in the daytime. The choice of an object height equal to the driver eye height makes passing sight distance design reciprocal (i.e., when the driver of the passing vehicle can see the opposing vehicle, the driver of the opposing vehicle can also see the passing vehicle).

Intersection sight distance object—As in the case of passing sight distance, the object to be seen by the driver in an intersection sight distance situation is another vehicle. Therefore, design for intersection sight distance is based on the same object height used in design for passing sight distance, 3.75 ft [1.14 m] ~~3.50 ft [1.08 m]~~.

Decision sight distance object—The 2.00-ft [0.60-m] object-height criterion adopted for stopping sight distance is also used for decision sight distance. The rationale for applying this object height for decision sight distance is the same as for stopping sight distance.

3.2.6.4 Measuring Sight Distance

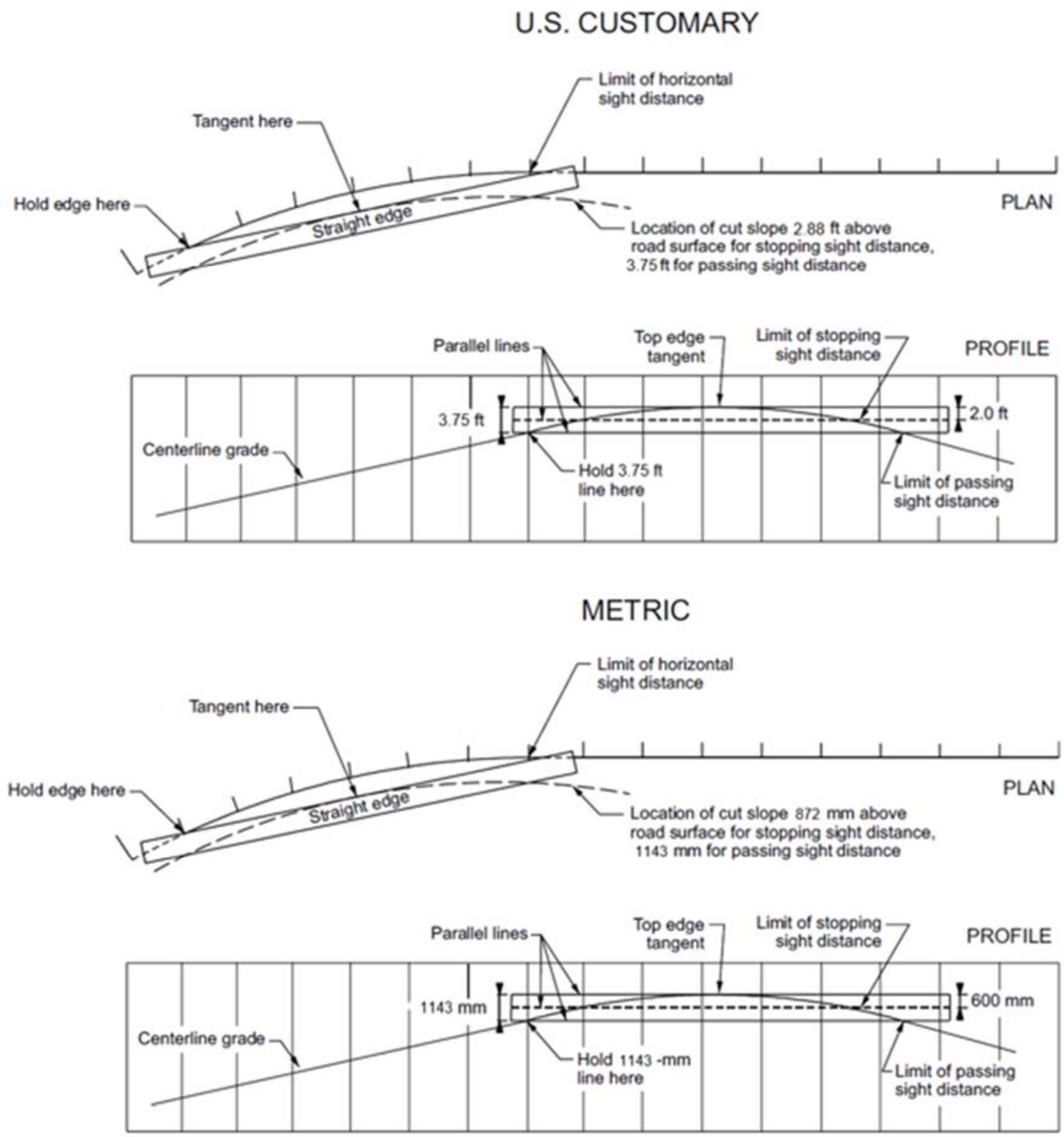
The design of horizontal alignment and vertical profile using sight distance and other criteria is addressed in Sections 3.3 through 3.5, including the detailed design of horizontal and vertical curves. Sight distance should be considered in the preliminary stages of design when both the horizontal and vertical alignment are still subject to adjustment. Stopping sight distance can easily be determined where plans and profiles are drawn using computer-aided design and drafting (CADD) systems. The line-of-sight that must be clear of obstructions is a straight line for the driver's eye position to an object on the road ahead, with the height of the driver's eye and the object as given above. The vertical component of sight distance is generally measured along the centerline of the roadway. The horizontal component of sight distance is normally measured along the centerline of the inside lane on a horizontal curve. By determining the available sight distances graphically on the plans and recording them at frequent intervals, the designer can review the overall layout and produce a more balanced design by minor adjustments in the plan or profile.

Because the view of the highway ahead may change rapidly in a short travel distance, it is desirable to measure and record sight distance for both directions of travel at each station. Both horizontal and vertical sight distances should be measured and the shorter lengths recorded. In the case of a two-lane highway, passing sight distance should be measured and recorded in addition to stopping sight distance.

Sight distance information, such as that presented in Figures 3-34 and 3-36 in Section 3.4.6, may be used to establish minimum lengths of vertical curves. Equation 3-37 can be used for determining the radius of horizontal curve or the lateral offset from the traveled way needed to provide the design sight distance. Examining sight distances along the proposed highway may be accomplished by measuring directly from the horizontal alignment and vertical profile in CADD systems. The following discussion presents a method for computing sight distances.

Horizontal sight distance on the inside of a curve is limited by obstructions such as buildings, hedges, wooded areas, high ground, or other topographic features. These are generally plotted on the plans. Horizontal sight distance is measured in CADD along a horizontal roadway alignment. Figure 3-1 illustrates the manual method for measuring sight distance, which is now automated in CADD systems. Preferably, the stopping sight distance should be measured between points on one traffic lane and passing sight distance from the middle of the other lane.

Such refinement on two-lane highways generally is not needed and measurement of sight distance along the centerline or traveled-way edge is suitable. Where there are changes of grade coincident with horizontal curves that have sight-limiting cut slopes on the inside, the line-of sight intercepts the slope at a level either lower or higher than the assumed average height. In measuring sight distance, the error in use of the assumed 2.88- or 3.75-ft [0.88- or 1.14-m] ~~2.75- or 3.50-ft [0.84- or 1.08-m]~~ height usually can be ignored.



Proposed Figure 3-1. Illustration of the Method for Measuring Sight Distance

Sight distance calculations for two-lane highways may be used effectively to tentatively determine the marking of no-passing zones in accordance with criteria given in the MUTCD (*24*). Marking of such zones is an operational rather than a design responsibility. No-passing zones thus established serve as a guide for markings when the highway is completed. The zones so determined should be checked and adjusted by field measurements before actual markings are placed.

Sight distance calculations also are useful on two-lane highways for determining the percentage of length of highway on which sight distance is restricted to less than the passing minimum, which is important in evaluating capacity.

3.3 Horizontal Alignment

3.3.3 Design Considerations

3.3.3.4 Effects of Grades

On long or fairly steep grades, drivers tend to travel faster in the downgrade than in the upgrade direction. Additionally, research (*16, 66*) has shown that the side friction demand is greater on both downgrades (due to braking forces) and steep upgrades (due to the tractive forces). Research (*66*) has also shown that, for simple horizontal curves, the maximum superelevation rate on steep downgrades of 4 percent or more should not exceed 12 percent. If considering a maximum superelevation rate on a horizontal curve in excess of 12 percent, a spiral curve transition is recommended to increase the margins of safety against skidding or rollover between the approach tangent and horizontal curve. Sharp horizontal curves (or near minimum-radius curves) on downgrades of 4 percent or more should not be designed using low design speeds (i.e., 30 mph [50 km/h] or less). In the event that such situations cannot be avoided, warning signs to reduce speeds well in advance of the start of the horizontal curve should be used.

On upgrades of 4 percent or more, the maximum superelevation rate should be limited to 9 percent for minimum-radius curves with design speeds of 55 mph [90 km/h] and higher, to minimize the potential for wheel-lift events on tractor semi-trailer trucks. Alternatively, if it can be verified that the available sight distance is such that deceleration at the rate assumed in stopping sight distance design criteria~~, 11.2 ft/s² [3.4 m/s²],~~ is unlikely to be needed on upgrades of 4 percent or more, e_{max} values up to 12 percent may be used for minimum-radius curves.

Vehicle dynamics simulations have shown (*66*) that sharp horizontal curves with near or minimum radii for given design speeds on downgrades of 4 percent or more could lead to skidding or rollover for a range of vehicle types if a driver is simultaneously braking and changing lanes on the curve. For this reason, it may be desirable to provide a "STAY IN LANE" sign (R4-9) in advance of sharp horizontal curves on steep grades on multilane highways (*24*). Consideration may also be given to using single solid white lane line markings to supplement the "STAY IN LANE" sign and discourage motorists from changing lanes.

3.3.8 Transition Design Controls

3.3.8.2 Tangent-to-Curve Transition

3.3.8.2.4 Limiting Superelevation Rates

Theoretical considerations indicate that, when a vehicle is traveling through a tangent-to-curve transition, large superelevation rates are associated with large shifts in the vehicle's lateral position. In general, such shifts in lateral position can be minimized by the proper location of the superelevation runoff section, as described above. However, large lateral shifts must be compensated by the driver through steering action.

In recognition of the potential adverse effect that large shifts in lateral position may have on vehicle control, the threshold superelevation rates associated with a lateral shift of 3.0 ft [1.0 m] are identified in Table 3-17. These limiting superelevation rates do not apply for speeds of 50 mph [80 km/h] or more when combined with superelevation rates of 12 percent or less.

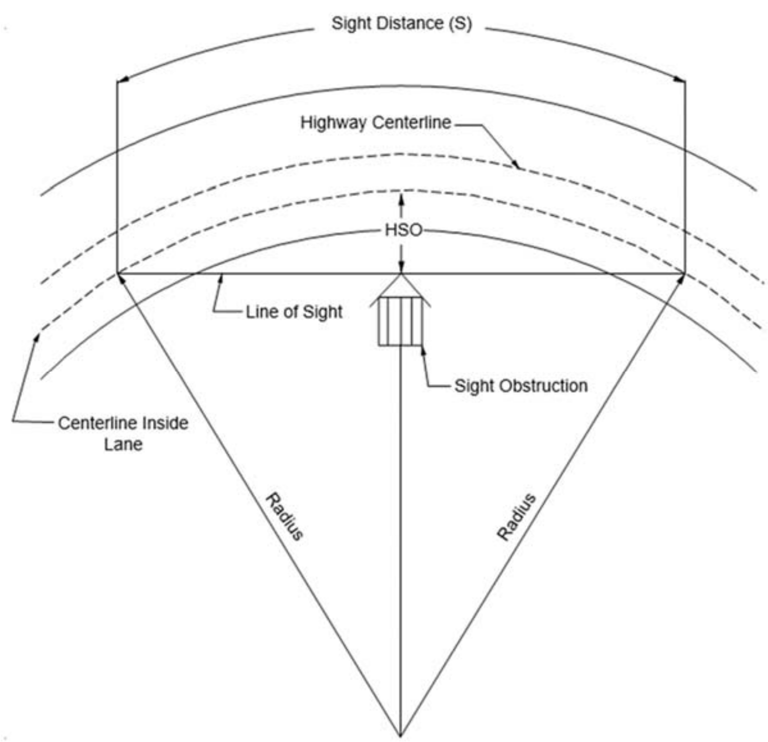
Designs that incorporate superelevation in excess of the limiting rates may be associated with excessive lateral shift. Therefore, it is recommended that such superelevation rates be avoided. However, if they are used, consideration should be given to increasing the width of the traveled way along the curve to reduce the potential for vehicle encroachment into the adjacent lane.

On upgrades of 4 percent or more, the maximum superelevation rate should be limited to 9 percent for minimum-radius curves with design speeds of 55 mph [90 km/h] and higher, to minimize the potential for wheel-lift events on tractor semi-trailer trucks. Alternatively, if it can be verified that the available sight distance is such that deceleration at the rate assumed in stopping sight distance design criteria~~, 11.2 ft/s~~$~~^2~~$ ~~[3.4 m/s~~$~~^2~~$~~],~~ is unlikely to be needed on upgrades of 4 percent or more, e_{max} values up to 12 percent may be used for minimum-radius curves (66).

3.3.12 Sight Distance on Horizontal Curves

3.3.12.1 Stopping Sight Distance

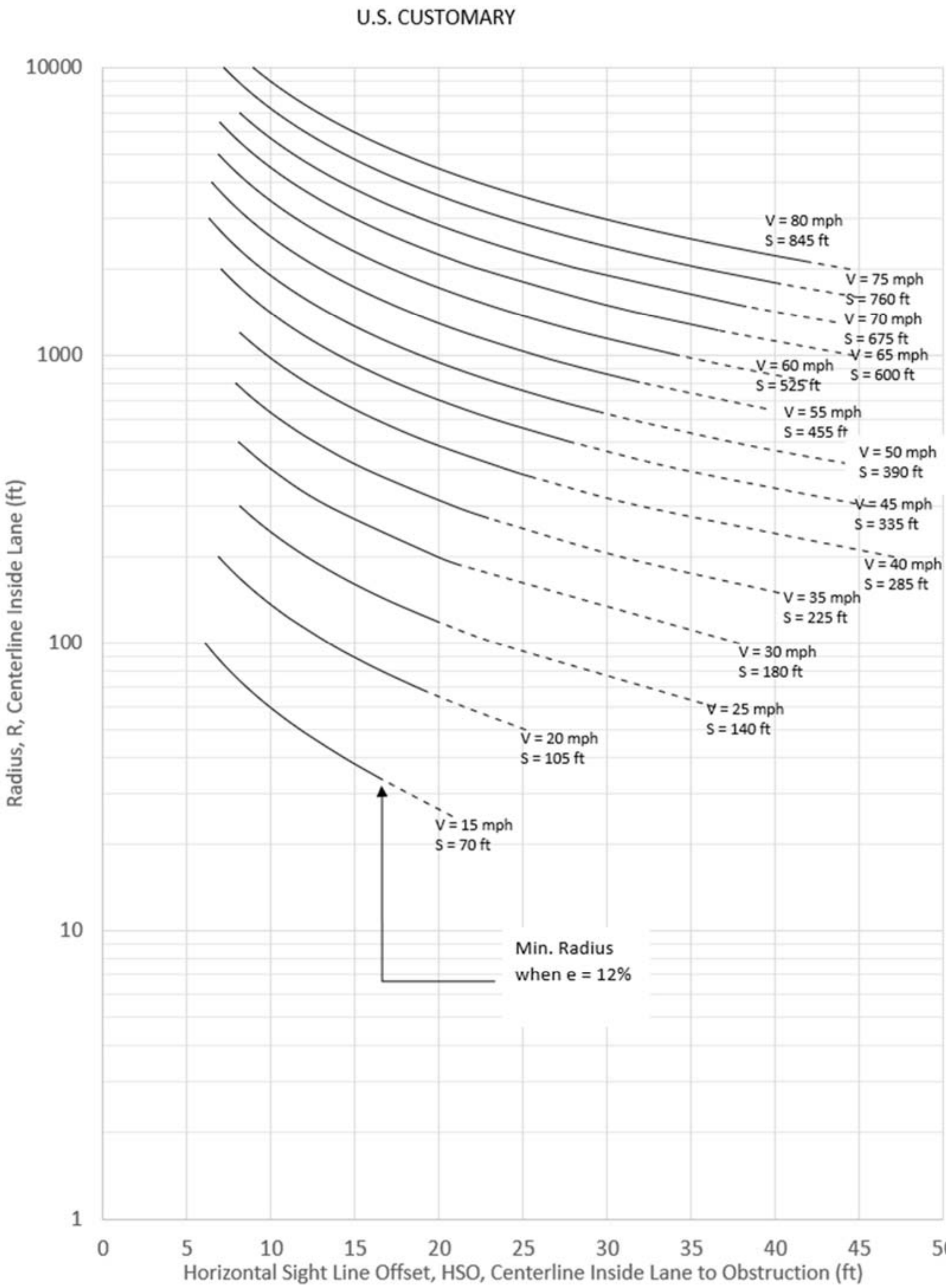
For general use in design of a horizontal curve, the sight line is a chord of the curve, and the stopping sight distance is measured along the centerline of the inside lane around the curve. The values of horizontal sight line offset (HSO) are determined by setting S, as shown in the diagrammatic sketch in Figure 3-13 and in Equation 3-37, equal to the stopping sight distance (SSD). Figure 3-14 shows the derived values of HSO. Equation 3-37 applies only to circular curves longer than the sight distance for the pertinent design speed. The relationships between R, HSO, and V in this chart can be quickly checked. For example, with a 50-mph [80-km/h] design speed and a curve with a 1,150-ft [350-m] radius, a clear sight area with a horizontal sight line offset of approximately 16.5 ft [5.0 m] ~~20 ft [6.0 m]~~ is needed for stopping sight distance. As another example, for a sight obstruction at a distance HSO equal to 16.5 ft [5.0 m] ~~20 ft [6.0 m]~~ from the centerline of the inside lane on a curve with a 575-ft [175-m] radius, the sight distance needed is approximately at the upper end of the range for a speed of approximately 40 mph [60 km/h] (assuming a rural context where 11.8 ft/s^2 [3.6 m/s^2] would apply).



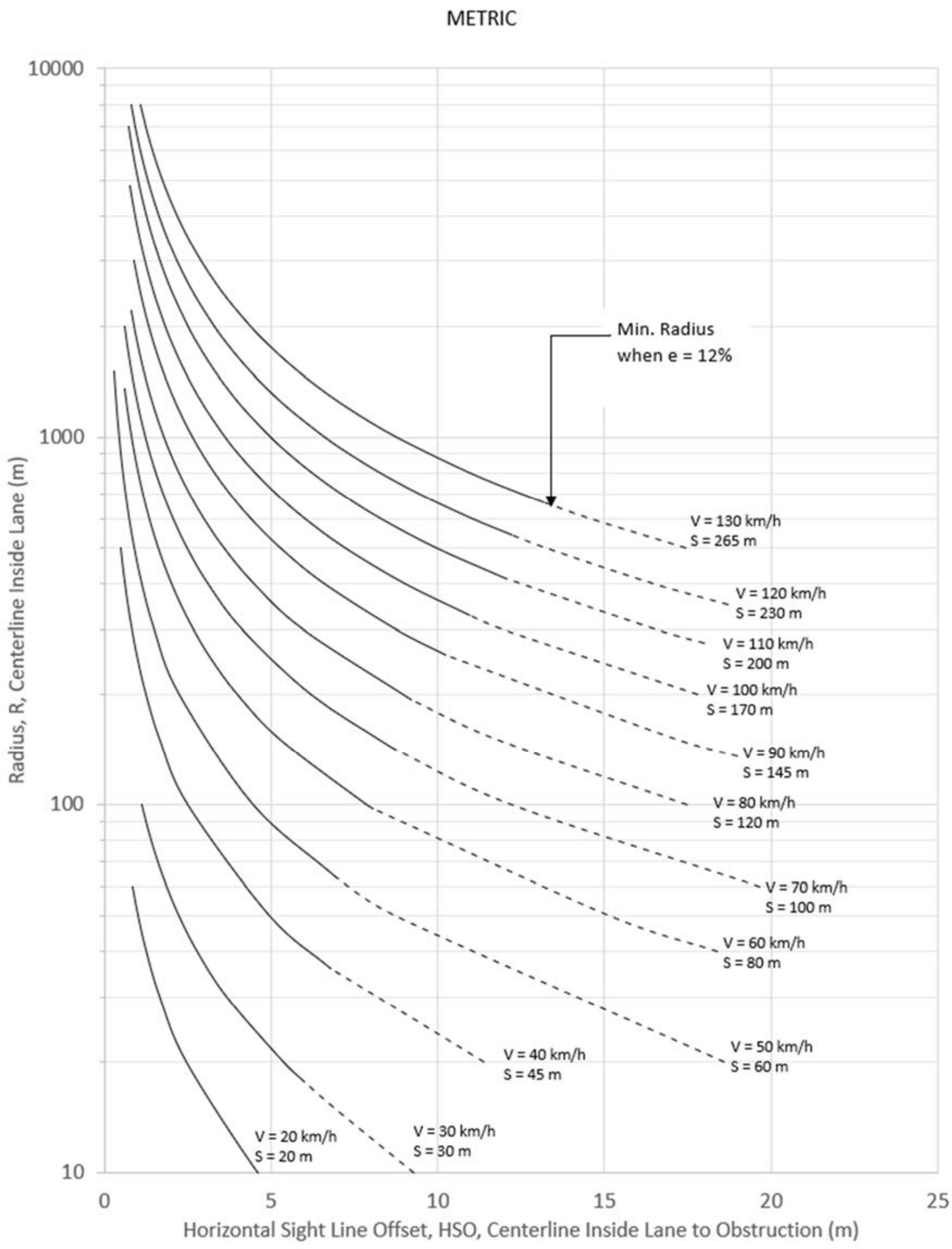
Source: *A Policy on Geometric Design of Highways and Streets* (2018) by AASHTO, Washington, D.C. Used with permission.

Figure 3-13. Diagram Illustrating Components for Determining Horizontal Sight Distance

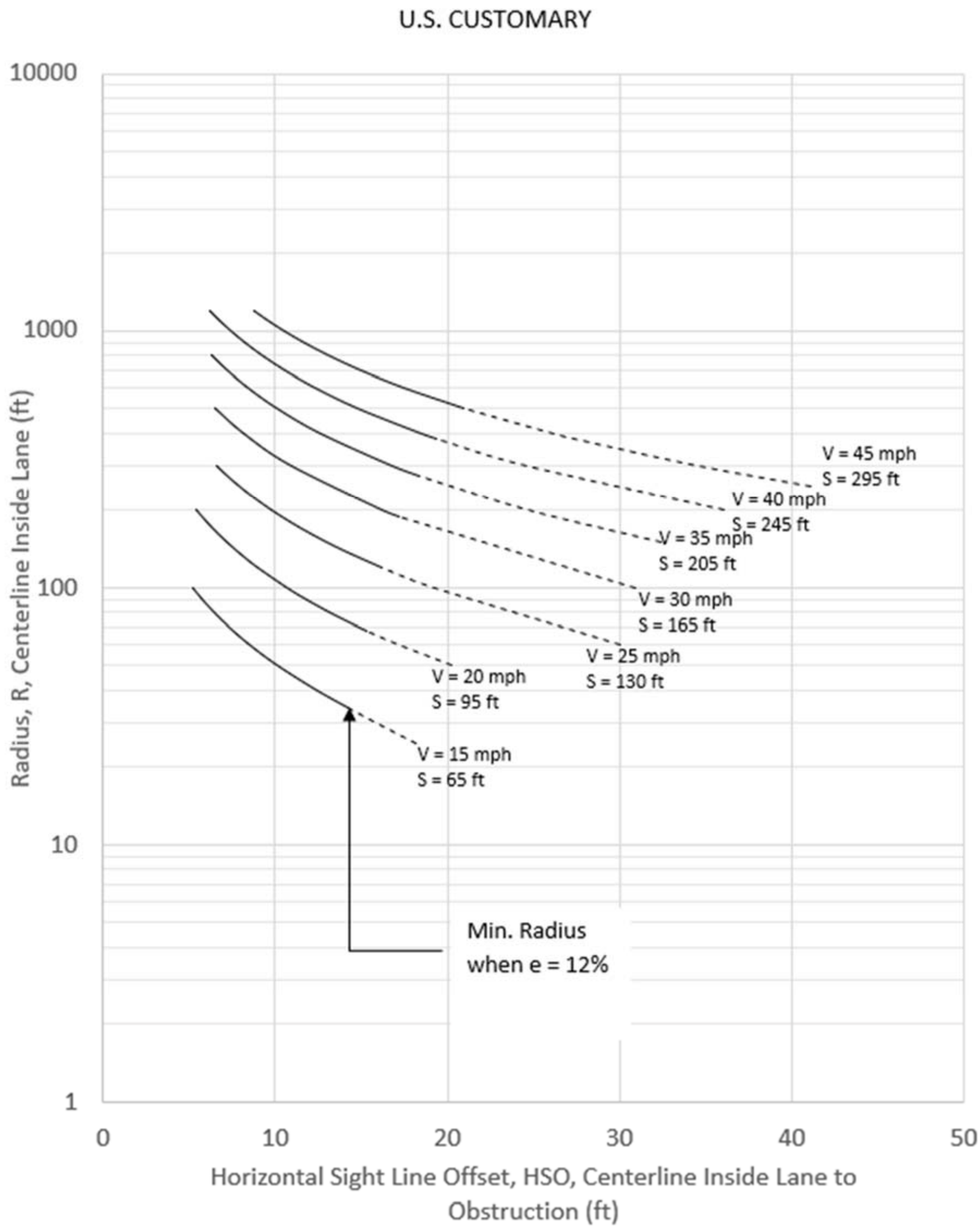
U.S. Customary	Metric	
$HSO = R\left[1-\cos\left(\frac{28.65S}{\boldsymbol{R}}\right)\right]$	$HSO = R\left[1-\cos\left(\frac{28.65S}{R}\right)\right]$	(3-37)
where:	where:	
HSO = Horizontal sight line offset, ft	HSO = Horizontal sight line offset, m	
S = Sight distance, ft	S = Sight distance, m	
R = Radius of curve, ft	R = Radius of curve, m	



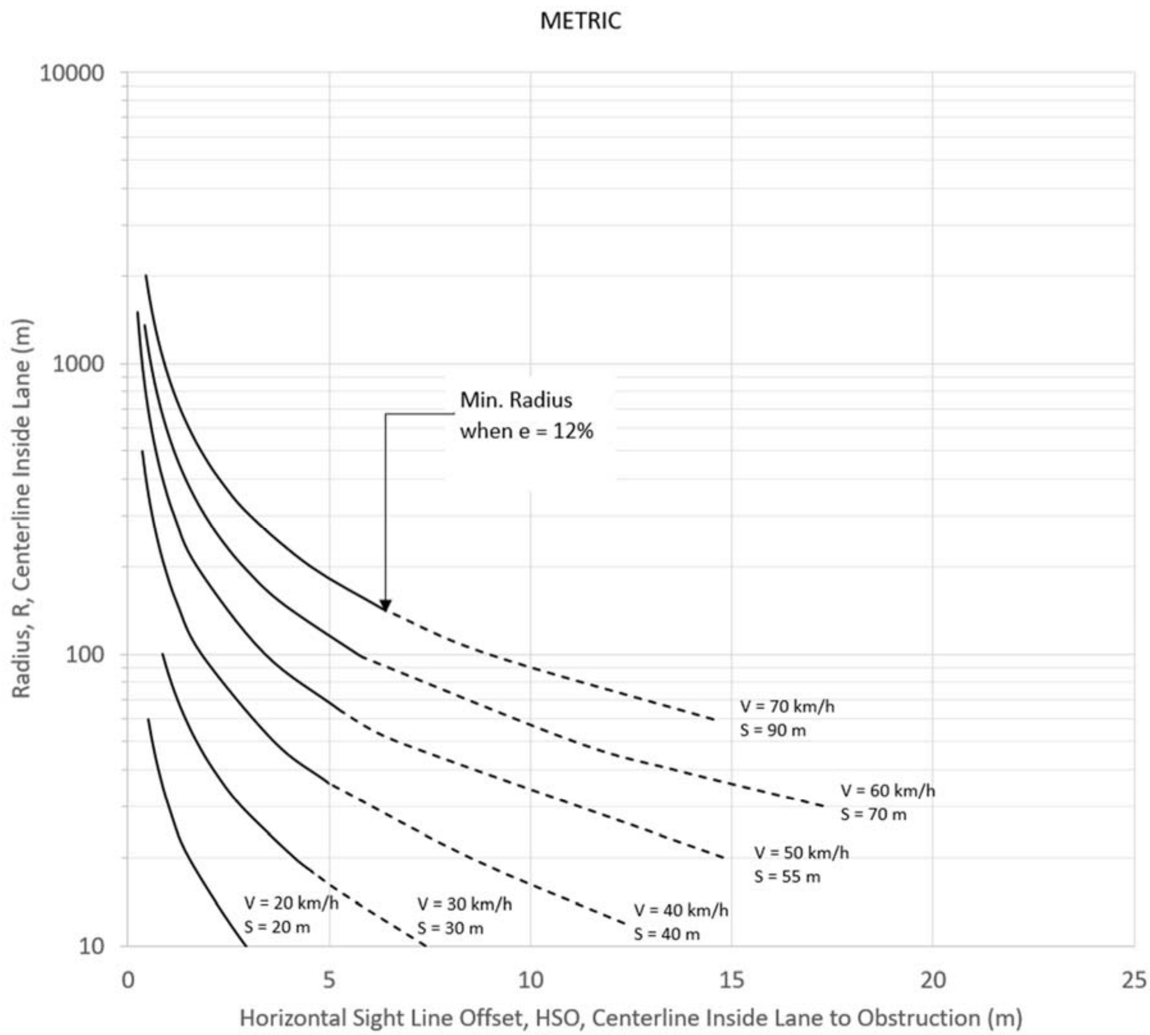
Proposed Figure 3-14. Horizontal Sightline Offset (HSO) to Provide Stopping Sight Distance on Horizontal Curves (RURAL OR HIGH SPEED)



Proposed Figure 3-14. Horizontal Sightline Offset (HSO) to Provide Stopping Sight Distance on Horizontal Curves (RURAL OR HIGH SPEED, Continued)



Proposed Figure 3-14. Horizontal Sightline Offset (HSO) to Provide Stopping Sight Distance on Horizontal Curves (LOW SPEED URBAN)



Proposed Figure 3-14. Horizontal Sightline Offset (HSO) to Provide Stopping Sight Distance on Horizontal Curves (LOW SPEED URBAN, Continued)

Horizontal sight restrictions may occur where there is a cut slope on the inside of the curve. For the 3.75 ft [1.14 m] ~~3.50 ft [1.08 m]~~ eye height and the 2.00-ft [0.60-m] object height used for stopping sight distance, a height of 2.88 ft [0.88 m] ~~2.75 ft [0.84 m]~~ may be used as the midpoint of the sight line where the cut slope usually obstructs sight. This assumes that there is little or no vertical curvature. For a highway with a 22 ft [6.6-m] traveled way, 4-ft [1.2-m] shoulders, an allowance of 4 ft [1.2 m] for a ditch section, and 1V:2H cut slopes (1 ft or 1 m vertically for each, 2 ft or 2 m horizontally), the sight obstruction is approximately 19 ft [5.75 m] outside the centerline of the inside lane. This is sufficient for adequate sight distance at 30 mph [50 km/h] when curves have a radius of about 225 ft [69 m] ~~275 ft [90 m]~~ or more (assuming a rural context where 11.8 ft/s^2 [3.6 m/s^2] would apply) and at 50 mph [80 km/h] when curves have a radius of about 1,000 ft [300 m] ~~1,230 ft [375 m]~~ or more. Curves sharper than these would need flatter slopes, benching, or other adjustments. At the other extreme, highways with normal lateral dimensions of more than 52 ft [16 m] provide adequate stopping sight distances for horizontal curves over the entire range of design speeds and curves.

In some instances, retaining walls, bridge rails, concrete median barriers, and other similar features constructed on the inside of curves may be sight obstructions and should be checked for stopping sight distance. As an example, an obstruction of this type, located 4 ft [1.2 m] from the inside edge of a 24-ft [7.2-m] traveled way, has a horizontal sight line offset of approximately 10 ft [3.0 m]. At 50 mph [80 km/h], this provides sufficient sight distance when a curve has a radius of about 1,900 ft [580 m] ~~2,300 ft [700 m]~~ or more. If the obstruction is moved an additional 1 ft [0.3 m] away from the roadway, creating a horizontal sight line offset of 11 ft [3.3 m], a curve with a radius of 1,700 ft [520 m] ~~2,000 ft [625 m]~~ or more provides sufficient sight distance at the same 50 mph [80 km/h] speed. The same finding would be applicable to existing buildings or similar sight obstructions on the inside of curves.

Where sufficient stopping sight distance is not available because a railing or a longitudinal barrier constitutes a sight obstruction, alternative designs should be considered. The alternatives are: (1) increase the offset to the obstruction, (2) increase the radius, or (3) reduce the design speed. However, the alternative selected should not incorporate shoulder widths on the inside of the curve in excess of 12 ft [3.6 m] because of the concern that drivers will use wider shoulders as a passing or travel lane.

As can be seen from Figure 3-14, the method presented is only exact when both the vehicle and the sight obstruction are located within the limits of the simple horizontal curve. When either the vehicle or the sight obstruction is situated beyond the limits of the simple curve, the values obtained are only approximate. The same is true if either the vehicle, the sight obstruction, or both are situated within the limits of a spiral or a compound curve. In these instances, the value obtained would result in horizontal sight line offset values slightly larger than those needed to satisfy the desired stopping sight distance. In many instances, the resulting additional clearance will not be significant. Whenever Figure 3-14 is not applicable, the design should be checked either by utilizing graphical procedures or by utilizing a computational method. Raymond (*52*) provides a computational method for making such checks.

Figure 3-14 is a design chart showing the horizontal sight line offsets needed for clear sight areas to provide the stopping sight distances presented in Table 3-1 for horizontal curves of various radii on flat grades. Figure 3-14 includes radii for all superelevation rates to a maximum of 12 percent. For the curves shown in Figure 3-14, the end of the solid line on the curve is the minimum radius where the superelevation is equal to 12 percent. The dashed portion of the curve is equal to values less than the standard minimum radius for a maximum superelevation rate of 12 percent.

3.3.12.2 Passing Sight Distance

The minimum passing sight distance for a two-lane road is about twice the minimum stopping sight distance at the same design speed. To conform to those greater sight distances, clear sight areas on the inside of curves should have widths in excess of those discussed. Equation 3-37 is directly applicable to passing sight distance but is of limited practical value except on long curves. A chart demonstrating use of this equation would primarily add value for reaching negative conclusions—that it would be difficult to maintain passing sight distance on other than very flat curves.

Passing sight distance is measured between an eye height of 3.75 ft [1.14 m] ~~3.50 ft [1.08 m]~~ and an object height of 3.75 ft [1.14 m] ~~3.50 ft [1.08 m]~~. This object height represents a near-maximum value for the portion of a passenger car height that needs to be visible for another driver to recognize it as such. The use of an object height equal to the driver eye height makes passing sight distances reciprocal (i.e., if one driver can see another vehicle, then the driver of that vehicle can also see the first vehicle).

The sight line near the center of the area inside a curve is approximately 0.88 ft [0.27 m] ~~0.75 ft [0.24 m]~~ higher than for stopping sight distance. In cut sections, the resultant lateral dimension for normal highway cross sections (1V:2H to 1V:6H backslopes) between the centerline of the inside lane and the midpoint of the sight line is from 1.5 to 4.5 ft [0.5 to 1.5 m] greater than that for stopping sight distance. It is obvious that for many cut sections, design for passing sight distance should, for practical reasons, be limited to tangents and very flat curves. Even in level terrain, provision of passing sight distance would need a clear area inside each curve that would, in some instances, extend beyond the normal right-of-way line.

In general, the designer should use graphical methods to check sight distance on horizontal curves. This method is presented in Figure 3-1 and described in the accompanying discussion.

3.4 Vertical Alignment

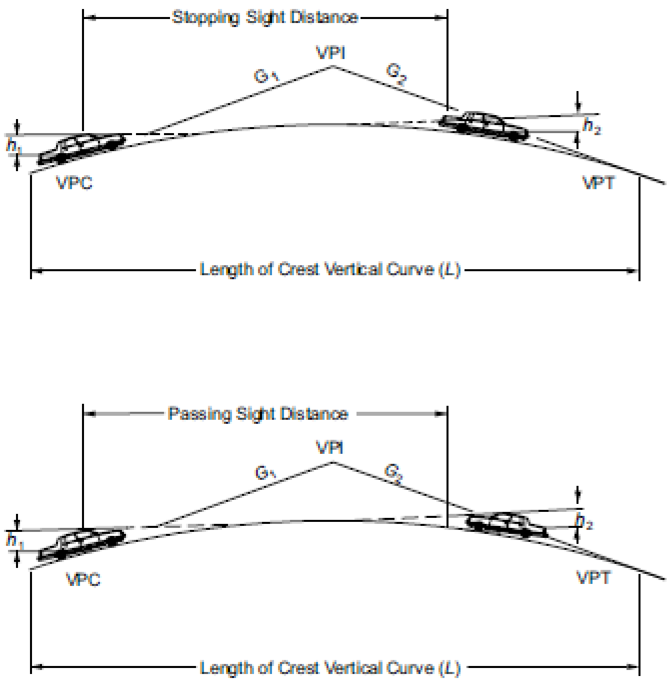
3.4.6 Vertical Curves

3.4.6.2 Crest Vertical Curves

Minimum lengths of crest vertical curves based on sight distance criteria generally are satisfactory from the standpoint of safety, comfort, and appearance. An exception may be at decision areas, such as ramp exit gores, where longer sight distances and, therefore, longer vertical curves should be provided; for further information, refer to Section 3.2.3, "Decision Sight Distance."

Figure 3-35 illustrates the parameters used in determining the length of a parabolic crest vertical curve needed to provide any specified value of sight distance. The basic equations for length of a crest vertical curve in terms of algebraic difference in grade and sight distance follow:

U.S. Customary	Metric	
When S is less than L, $L = \frac{AS^2}{100\left(\sqrt{2h_1}+\sqrt{2h_2}\right)^2}$	When S is less than L, $L = \frac{AS^2}{100\left(\sqrt{2h_1}+\sqrt{2h_2}\right)^2}$	(3-42)
When S is greater than L, $L = 2S - \frac{200\left(\sqrt{h_1}+\sqrt{h_2}\right)^2}{A}$	When S is greater than L, $L = 2S - \frac{200\left(\sqrt{h_1}+\sqrt{h_2}\right)^2}{A}$	(3-43)
where:	where:	
L = length of vertical curve, ft	L = length of vertical curve, m	
A = algebraic difference in grades, percent	A = algebraic difference in grades, percent	
S = sight distance, ft	S = sight distance, m	
h_1 = height of eye above roadway surface, ft	h_1 = height of eye above roadway surface, m	
h_2 = height of object above roadway surface, ft	h_2 = height of object above roadway surface, m	



Source: *A Policy on Geometric Design of Highways and Streets* (2018) by AASHTO, Washington, D.C. Used with permission.

Figure 3-35. Parameters Considered in Determining the Length of a Crest Vertical Curve to Provide Sight Distance

When the height of eye and the height of object are 3.75 ft [1.14 m] ~~3.50 ft [1.08 m]~~ and 2.00 ft [1.08 and 0.60 m], respectively, as used for stopping sight distance, the equations become:

U.S. Customary	Metric	
When S is less than L, $L = \frac{AS^2}{2{,}245\ \cancel{2158}}$	When S is less than L, $L = \frac{AS^2}{679\ \cancel{658}}$	(3-44)
When S is greater than L, $L = 2S - \frac{2{,}245\ \cancel{2158}}{A}$	When S is greater than L, $L = 2S - \frac{679\ \cancel{658}}{A}$	(3-45)

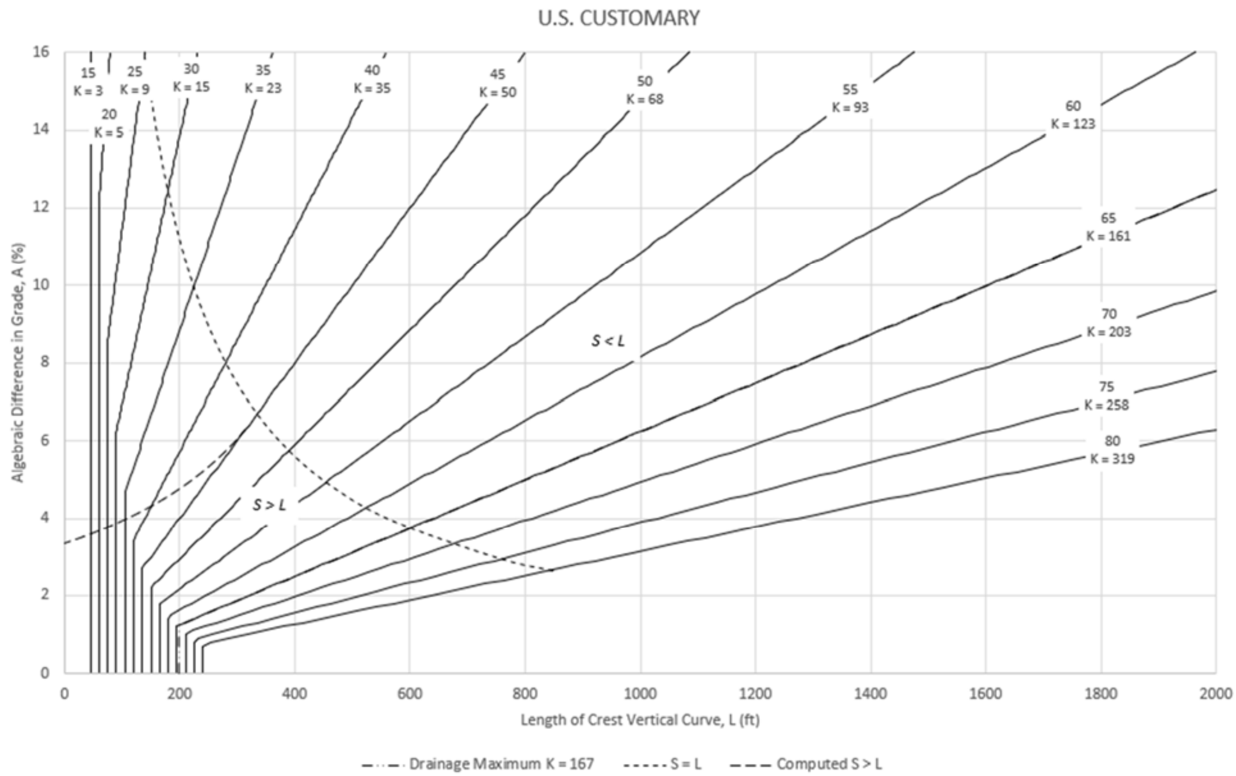
3.4.6.2.1 Design Controls: Stopping Sight Distance

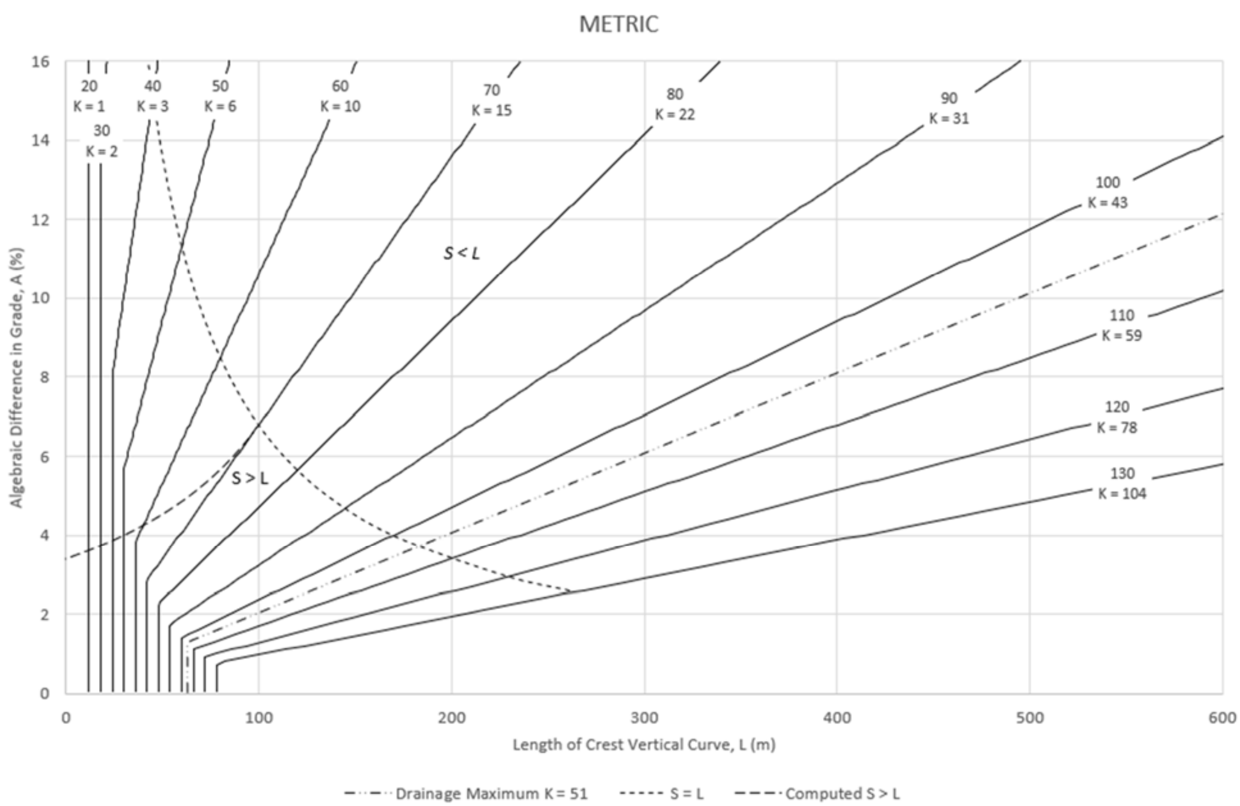
The minimum lengths of crest vertical curves for different values of A to provide the minimum stopping sight distances for each design speed are shown in Figure 3-36. The solid lines give the minimum vertical curve lengths, on the basis of rounded values of K as determined from Equations 3-44 and 3-45.

The short dashed curve at the lower left, crossing these lines, indicates where $S = L$. Note that to the right of the $S = L$ line, the value of K, or length of vertical curve per percent change in A, is a simple and convenient expression of the design control. For each design speed, this single value is a positive whole number that is indicative of the rate of vertical curvature. The design control in terms of K covers all combinations of A and L for any one design speed; thus, A and L need not be indicated separately in a tabulation of design value. The selection of design curves is facilitated because the minimum length of curve in feet [meters] is equal to K times the algebraic difference in grades in percent, $L = KA$. Conversely, the checking of plans is simplified by comparing all curves with the design value for K.

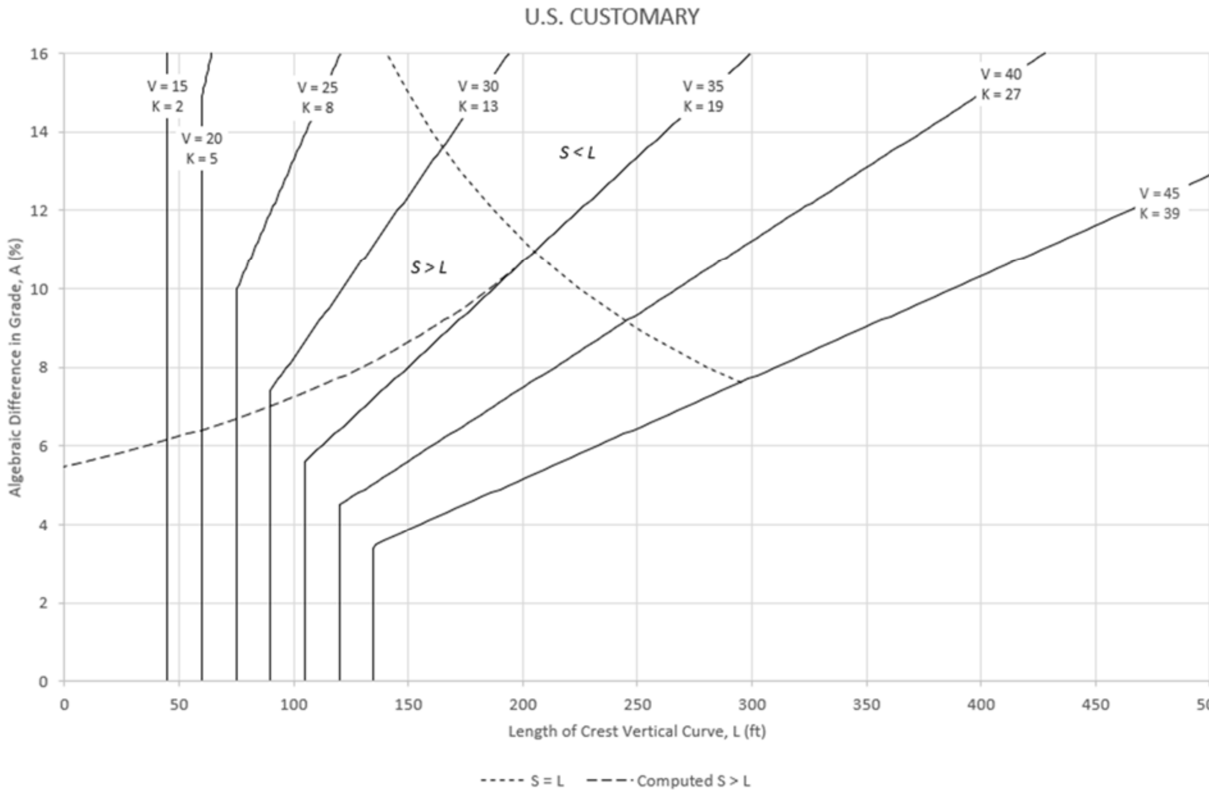
Table 3-35 shows the computed K values for lengths of vertical curves corresponding to the stopping sight distances shown in Table 3-1 for each design speed. For direct use in design, values of K are rounded as shown in the right column. The rounded values of K are plotted as the solid lines in Figure 3-36. These rounded values of K are higher than computed values, but the differences are not significant.

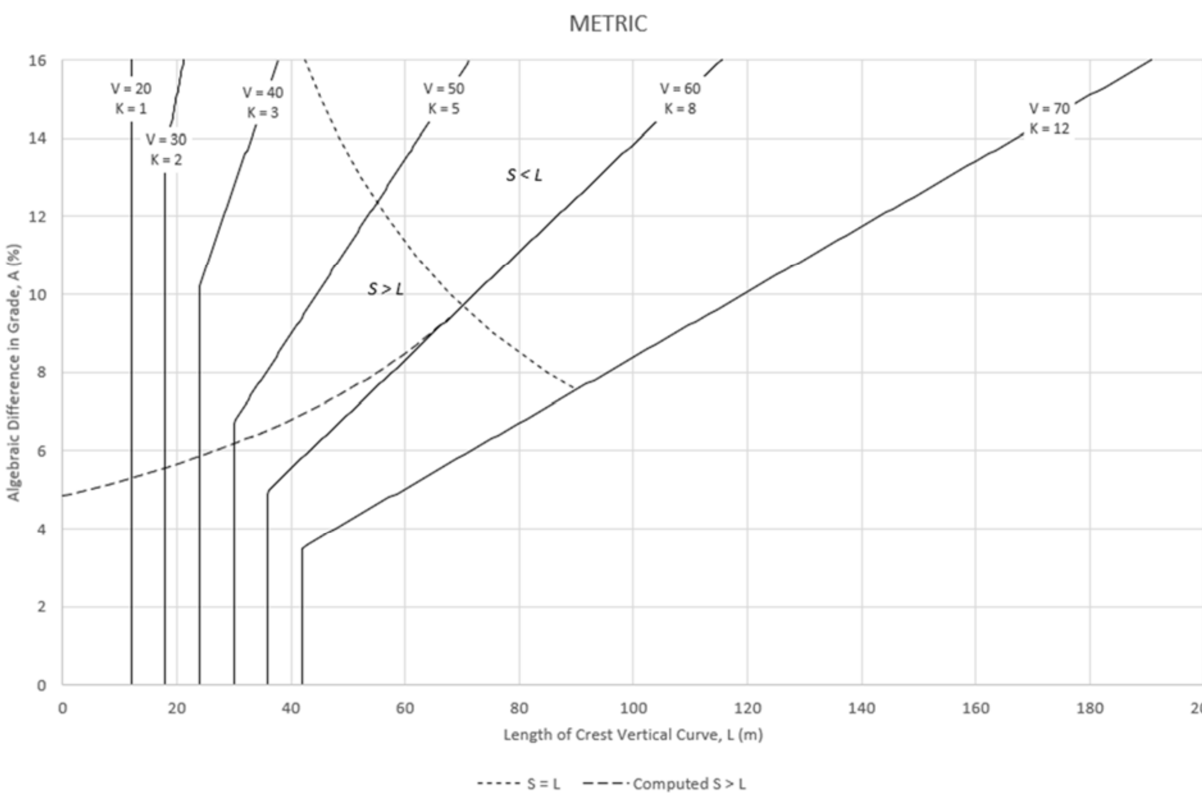
Where S is greater than L (lower left in Figure 3-36), the computed values plot as a curve (as shown by the dashed line for 45 mph [70 km/h]) that bends to the left, and for small values of A, the vertical curve lengths are zero because the sight line passes over the high point. This relationship does not represent desirable design practice. Most states use a minimum length of vertical curve, expressed as a single value, a range for different design speeds, or a function of A. Values now in use range from about 100 to 325 ft [30 to 100 m]. To recognize the distinction in design speed and to approximate the range of current practice, minimum lengths of vertical curves are expressed as about 0.6 times the design speed in km/h, $L_{min} = 0.6V$, where V is in kilometers per hour and L is in meters, or about three times the design speed in mph, [$L_{min} = 3V$], where V is in miles per hour and L is in feet. These terminal adjustments show as the vertical lines at the lower left of Figure 3-36.





Proposed Figure 3-36. Design Controls for Crest Vertical Curves- Open Road Conditions (RURAL OR HIGH SPEED)





Proposed Figure 3-36. Design Controls for Crest Vertical Curves- Open Road Conditions (LOW SPEED URBAN)

~~Table 3-35. Design Controls for Crest Vertical Curves Based on Stopping Sight Distance~~

~~U.S. Customary~~			
~~Design Speed (mph)~~	~~Stopping Sight Distance (ft)~~	~~Rate of Vertical Curvature, Ka~~	
		~~Calculated~~	~~Design~~
~~15~~	~~80~~	~~3.0~~	~~3~~
~~20~~	~~115~~	~~6.1~~	~~7~~
~~25~~	~~155~~	~~11.1~~	~~12~~
~~30~~	~~200~~	~~18.5~~	~~19~~
~~35~~	~~250~~	~~29.0~~	~~29~~
~~40~~	~~305~~	~~43.1~~	~~44~~
~~45~~	~~360~~	~~60.1~~	~~61~~
~~50~~	~~425~~	~~83.7~~	~~84~~
~~55~~	~~495~~	~~113.5~~	~~114~~
~~60~~	~~570~~	~~150.6~~	~~151~~
~~65~~	~~645~~	~~192.8~~	~~193~~
~~70~~	~~730~~	~~246.9~~	~~247~~
~~75~~	~~820~~	~~311.6~~	~~312~~
~~80~~	~~910~~	~~383.7~~	~~384~~

~~Metric~~			
~~Design Speed (km/h)~~	~~Stopping Sight Distance (m)~~	~~Rate of Vertical Curvature, Ka~~	
		~~Calculated~~	~~Design~~
~~20~~	~~20~~	~~0.6~~	~~1~~
~~30~~	~~35~~	~~1.9~~	~~2~~
~~40~~	~~50~~	~~3.8~~	~~4~~
~~50~~	~~65~~	~~6.4~~	~~7~~
~~60~~	~~85~~	~~11.0~~	~~11~~
~~70~~	~~105~~	~~16.8~~	~~17~~
~~80~~	~~130~~	~~25.7~~	~~26~~
~~90~~	~~160~~	~~38.9~~	~~39~~
~~100~~	~~185~~	~~52.0~~	~~52~~
~~110~~	~~220~~	~~73.6~~	~~74~~
~~120~~	~~250~~	~~95.0~~	~~95~~
~~130~~	~~285~~	~~123.4~~	~~124~~

Source: *A Policy on Geometric Design of Highways and Streets* (2018) by AASHTO, Washington, D.C. Used with permission.

Proposed Table 3-35. Design Controls for Crest Vertical Curves Based on SSD RURAL OR HIGH SPEED

U.S. Customary				Metric			
Design Speed (mph)	Stopping Sight Distance (ft)	Rate of Vertical Curvature, Ka		Design Speed (km/h)	Stopping Sight Distance (m)	Rate of Vertical Curvature, Ka	
		Calculated	Design			Calculated	Design
15	70	2.2	3	20	20	0.6	1
20	105	4.9	5	30	30	1.3	2
25	140	8.7	9	40	45	3.0	3
30	180	14.4	15	50	60	5.3	6
35	225	22.6	23	60	80	9.4	10
40	280	34.9	35	70	100	14.7	15
45	335	50.0	50	80	120	21.2	22
50	390	67.8	68	90	145	31.0	31
55	455	92.2	93	100	170	42.6	43
60	525	122.8	123	110	200	58.9	59
65	600	160.4	161	120	230	77.9	7
70	675	203.0	203	130	265	103.4	104
75	760	257.3	258				
80	845	318.1	319				

[a] Rate of vertical curvature, *K*, is the length of curve per percent algebraic difference in intersecting grades (*A*), *K* = *L*/*A*.

LOW SPEED URBAN U.S. Customary				Metric			
Design Speed (mph)	Stopping Sight Distance (ft)	Rate of Vertical Curvature, Ka		Design Speed (km/h)	Stopping Sight Distance (m)	Rate of Vertical Curvature, Ka	
		Calculated	Design			Calculated	Design
15	65	1.9	2	20	20	0.6	1
20	95	4.0	5	30	30	1.3	2
25	130	7.5	8	40	40	2.4	3
30	165	12.1	13	50	55	4.5	5
35	205	18.7	19	60	70	7.2	8
40	245	26.7	27	70	90	11.9	12
45	295	38.8	39				

[a] Rate of vertical curvature, *K*, is the length of curve per percent algebraic difference in intersecting grades (*A*), *K* = *L*/*A*.

The values of K derived above when S is less than L also can be used without significant error where S is greater than L. As shown in Figure 3-35, extension of the diagonal lines to meet the vertical lines for minimum lengths of vertical curves results in appreciable differences from the theoretical only where A is small and little or no additional cost is involved in obtaining longer vertical curves.

For night driving on highways without lighting, the length of visible roadway is that roadway that is directly illuminated by the headlights of the vehicle. For certain conditions, the minimum stopping sight distance values used for design exceed the length of visible roadway. First, vehicle headlights have limitations on the distance over which they can project the light intensity levels that are needed for visibility. When headlights are operated on low beams, the reduced candlepower at the source plus the downward projection angle significantly restrict the length of visible roadway surface. Thus, particularly for high-speed conditions, stopping sight distance values exceed road-surface visibility distances afforded by the low-beam headlights regardless of whether the roadway profile is level or curving vertically. Second, for crest vertical curves, the area forward of the headlight beam's point of tangency with the roadway surface is shadowed and receives only indirect illumination.

Since the headlight mounting height (typically about 2.00 ft [0.60 m]) is lower than the driver eye height used for design (3.75 ft [1.14 m] ~~3.50 ft [1.08 m]~~), the sight distance to an illuminated object is controlled by the height of the vehicle headlights rather than by the direct line of sight. Any object within the shadow zone must be high enough to extend into the headlight beam to be directly illuminated. On the basis of Equation 3-41, the bottom of the headlight beam is about 1.30 ft [0.40 m] above the roadway at a distance ahead of the vehicle equal to the stopping sight distance. Although the vehicle headlight system does limit roadway visibility length as previously mentioned, there is some mitigating effect in that other vehicles, whose taillight height typically varies from 2.50 to 3.50 ft [0.75 to 1.05 m] ~~1.50 to 2.00 ft [0.45 to 0.60 m]~~, and other sizable objects receive direct lighting from headlights at stopping sight distance values used for design. Furthermore, drivers are aware that visibility at night is less than during the day, regardless of road and street design features, and they may therefore be more attentive and alert.

There is a level point on a crest vertical curve of Type I (see Figure 3-34), but no difficulty with drainage on highways with curbs is typically experienced if the curve is sharp enough so that a minimum grade of 0.30 percent is reached at a point about 50 ft [15 m] from the crest. This corresponds to *K* of 167 ft [51 m] per percent change in grade, which is plotted in Figure 3-36 as the drainage maximum. All combinations above or to the left of this line satisfy the drainage criterion. The combinations below and to the right of this line involve flatter vertical curves. Special attention is needed in these cases to provide proper pavement drainage near the high point of crest vertical curves. It is not intended that K of 167 ft [51 m] per percent grade be considered a design maximum, but merely a value beyond which drainage should be more carefully designed.

3.4.6.2.2 Design Controls: Passing Sight Distance

Design values of crest vertical curves for passing sight distance differ from those for stopping sight distance because of the different sight distance and object height criteria. The general Equations 3-42 and 3-43 apply. Using the 3.75 ft [1.14 m] ~~3.50 ft [1.08 m]~~ height of object results in the following specific formulas with the same terms as shown above:

U.S. Customary	Metric	
When S is less than L, $L = \frac{AS^2}{3{,}000\ \cancel{2800}}$	When S is less than L, $L = \frac{AS^2}{912\ \cancel{864}}$	(3-46)
When S is greater than L, $L = 2S - \frac{3{,}000\ \cancel{2800}}{A}$	When S is greater than L, $L = 2S - \frac{912\ \cancel{864}}{A}$	(3-47)

For the minimum passing sight distances shown in Table 3-4, the minimum lengths of crest vertical curves are substantially longer than those for stopping sight distances. The extent of difference is evident by the values of K, or length of vertical curve per percent change in A, for passing sight distances shown in Table 3-36.

~~***Table 3-36. Design Controls for Crest Vertical Curves Based on Passing Sight Distance***~~

~~U.S. Customary~~			~~Metric~~		
~~Design Speed (mph)~~	~~Passing Sight Distance (ft)~~	~~Rate of Vertical Curvature, Ka Design~~	~~Design Speed (km/h)~~	~~Passing Sight Distance (m)~~	~~Rate of Vertical Curvature, Ka Design~~
~~20~~	~~400~~	~~57~~	~~30~~	~~120~~	~~17~~
~~25~~	~~450~~	~~72~~	~~40~~	~~140~~	~~23~~
~~30~~	~~500~~	~~89~~	~~50~~	~~160~~	~~30~~
~~35~~	~~550~~	~~108~~	~~60~~	~~180~~	~~38~~
~~40~~	~~600~~	~~129~~	~~70~~	~~210~~	~~51~~
~~45~~	~~700~~	~~175~~	~~80~~	~~245~~	~~69~~
~~50~~	~~800~~	~~229~~	~~90~~	~~280~~	~~91~~
~~55~~	~~900~~	~~289~~	~~100~~	~~320~~	~~119~~
~~60~~	~~1000~~	~~357~~	~~110~~	~~355~~	~~146~~
~~65~~	~~1100~~	~~432~~	~~120~~	~~395~~	~~181~~
~~70~~	~~1200~~	~~514~~	~~130~~	~~440~~	~~224~~
~~75~~	~~1300~~	~~604~~			
~~80~~	~~1400~~	~~700~~			

~~[a] Rate of vertical curvature, *K*, is the length of curve per percent algebraic difference in intersecting grades (*A*), *K* = *L/A*.~~

Source: *A Policy on Geometric Design of Highways and Streets* (2018) by AASHTO, Washington, D.C. Used with permission.

Proposed Table 3-36. Design Controls for Crest Vertical Curves Based on Passing Sight Distance

U.S. Customary			Metric		
Design Speed (mph)	Passing Sight Distance (ft)	Rate of Vertical Curvature, Ka Design	Design Speed (km/h)	Passing Sight Distance (m)	Rate of Vertical Curvature, Ka Design
20	400	54	30	120	16
25	450	68	40	140	22
30	500	84	50	160	29
35	550	101	60	180	36
40	600	120	70	210	49
45	700	164	80	245	66
50	800	214	90	280	86
55	900	270	100	320	113
60	1000	334	110	355	139
65	1100	404	120	395	172
70	1200	480	130	440	213
75	1300	564			
80	1400	654			

[a] Rate of vertical curvature, *K*, is the length of curve per percent algebraic difference in intersecting grades (*A*), *K* = *L/A*.

Generally, it is impractical to design crest vertical curves that provide passing sight distance because of high cost where crest cuts are involved and the difficulty of fitting the resulting long vertical curves to the terrain, particularly for high-speed roads. Passing sight distance on crest vertical curves may be practical on roads with unusual combinations of low design speeds and gentle grades or higher design speeds with very small algebraic differences in grades. Ordinarily, passing sight distance is provided only at locations where combinations of alignment and profile do not need significant grading. Table 3-36 shows computed K values for determining lengths of vertical curves corresponding to passing sight distance values shown in Table 3-4.

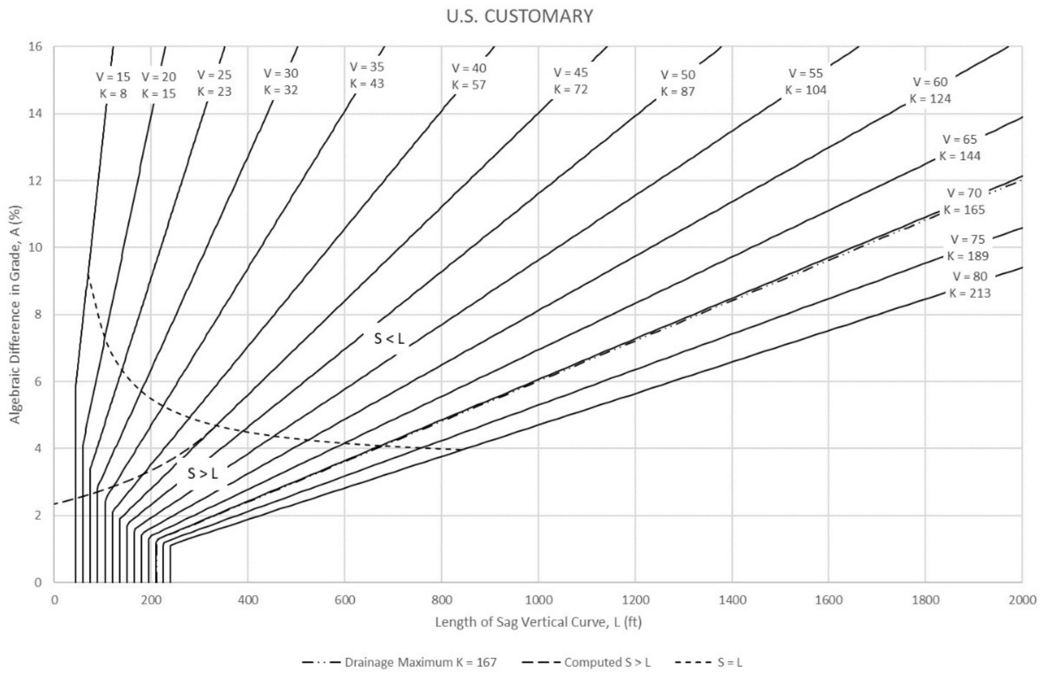
3.4.6.3 Sag Vertical Curves

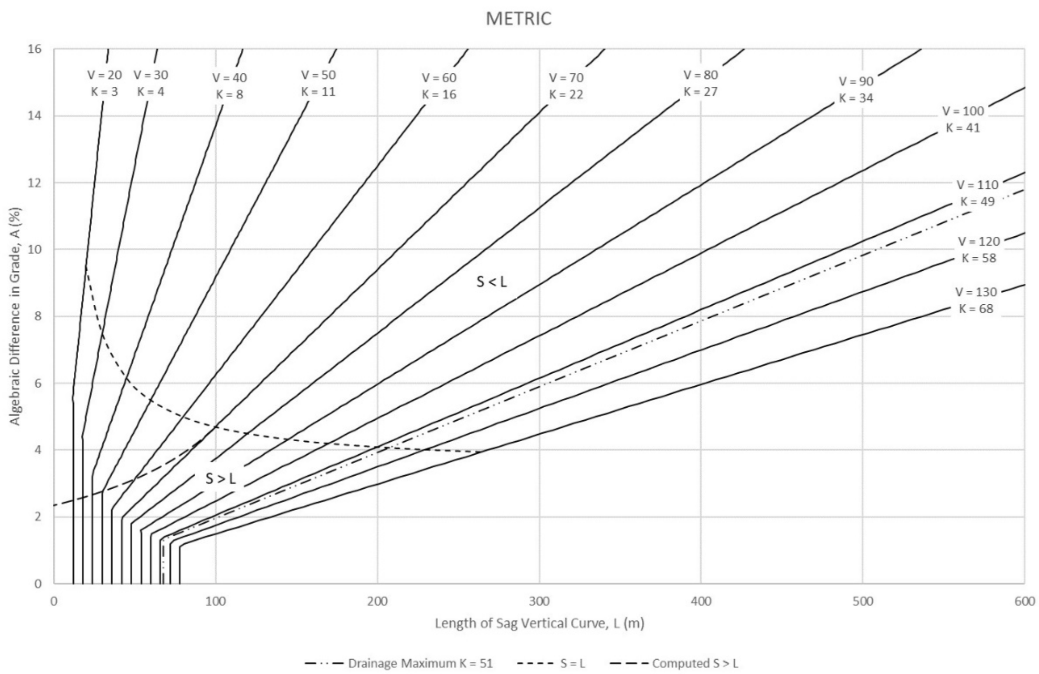
At least four different criteria for establishing lengths of sag vertical curves are recognized to some extent. These are (1) headlight sight distance, (2) passenger comfort, (3) drainage control, and (4) general appearance.

Headlight sight distance has been used directly by some agencies and for the most part is the basis for determining the desirable length of sag vertical curves. When a vehicle traverses a sag vertical curve at night, the portion of highway lighted ahead is dependent on the position of the headlights and the direction of the light beam. A headlight height of 2 ft [0.60 m] and a 1-degree upward divergence of the light beam from the longitudinal axis of the vehicle is commonly assumed. The upward spread of the light beam above the 1-degree divergence angle provides some additional visible length of roadway. For sag vertical curves without an overhead vertical restriction, drivers can utilize high beams, highway lighting, or the lights from other vehicles. The following equations show the relationships between S, L, and A, using S as the distance between the vehicle and point where the 1-degree upward angle of the light beam intersects the surface of the roadway:

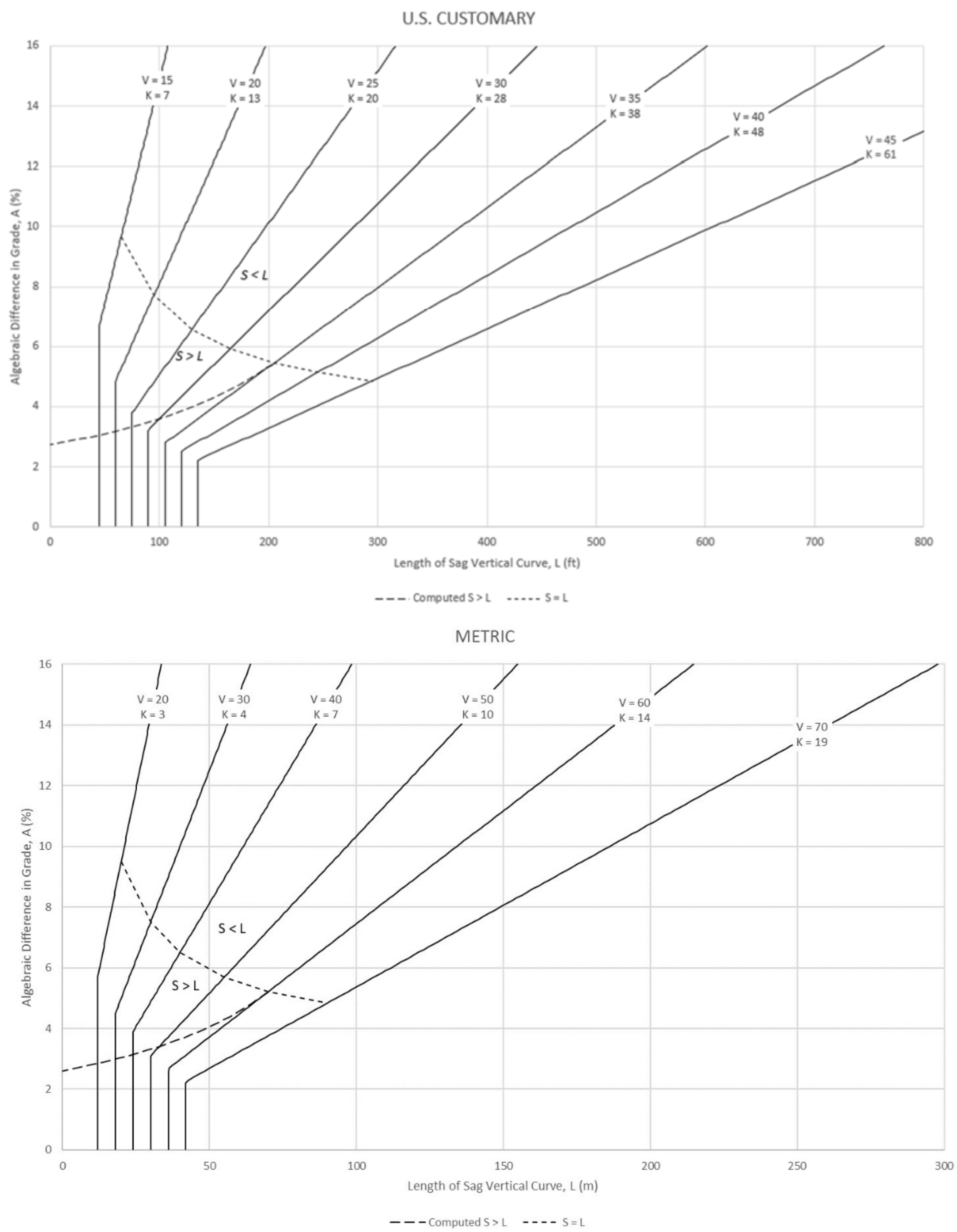
U.S. Customary	Metric	
When S is less than L, $L = \frac{AS^2}{200[2.0 + S(\tan 1^o)]}$	When S is less than L, $L = \frac{AS^2}{200[0.6 + S(\tan 1^o)]}$	(3-48)
or, $L = \frac{AS^2}{400 + 3.5S}$	or, $L = \frac{AS^2}{120 + 3.5S}$	(3-49)
When S is greater than L, $L = 2S - \frac{200[2.0 + S(\tan 1^o)]}{A}$	When S is greater than L, $L = 2S - \frac{200[0.6 + S(\tan 1^o)]}{A}$	(3-50)
or, $L = 2S - \frac{400 + 3.5S}{A}$	or, $L = 2S - \frac{120 + 3.5S}{A}$	(3-51)
where: L = length of vertical curve, ft A = algebraic difference in grades, percent S = sight distance, ft	where: L = length of vertical curve, m A = algebraic difference in grades, percent S = sight distance, m	

It is desirable that a sag vertical curve be long enough that the light beam distance is approximately the same as the stopping sight distance. Accordingly, it is appropriate to use stopping sight distances for different design speeds as the value of S in the above equations. The resulting lengths of sag vertical curves for the desirable stopping sight distances for each design speed are shown in Figure 3-37 with solid lines using rounded values of K as was done for crest vertical curves.





Proposed Figure 3-37. Design Controls for Sag Vertical Curves- Open Road Conditions (RURAL OR HIGH SPEED)



Proposed Figure 3-37. Design Controls for Sag Vertical Curves- Open Road Conditions (LOW SPEED URBAN)

The effect on passenger comfort of the change in vertical direction is greater on sag than on crest vertical curves because gravitational and centripetal forces are combining rather than opposing forces. Comfort due to change in vertical direction is not easily measured because it is affected appreciably by vehicle body suspension, vehicle body weight, tire flexibility, and other factors. Limited attempts at such measurements have led to the broad conclusion that riding is comfortable on sag vertical curves when the centripetal acceleration does not exceed 1 ft/s^2 [0.3 m/s^2]. The general expression for such a criterion is:

U.S. Customary	Metric	
$L = \frac{AV^2}{46.5}$	$L = \frac{AV}{395}$	(3-52)
where: L = length of vertical curve, ft A = algebraic difference in grades, percent V = design speed, mph	where: L = length of vertical curve, m A = algebraic difference in grades, percent V = design speed, km/h	

The length of vertical curve needed to satisfy this comfort factor at the various design speeds is only about 50 percent of that needed to satisfy the headlight sight distance criterion for the normal range of design conditions.

Drainage affects design of vertical curves of Type III (see Figure 3-35) where curbed sections are used. An approximate criterion for sag vertical curves is the same as that expressed for the crest conditions (i.e., a minimum grade of 0.30 percent should be provided within 50 ft [15 m] of the level point). This criterion corresponds to K of 167 ft [51 m] per percent change in grade, which is plotted in Figure 3-37 as the drainage maximum. The drainage criterion differs from other criteria in that the length of sag vertical curve determined for it is a maximum, whereas the length for any other criterion is a minimum. The maximum length of the drainage criterion is greater than the minimum length for other criteria up to ~~65~~ 70 mph [~~100~~ 110 km/h].

For improved appearance of sag vertical curves, previous guidance used a rule-of-thumb for minimum curve length of 30A [100A] or, in Figure 3-37, K = 100 ft [K = 30 m] per percent change in grade. This approximation is a generalized control for small or intermediate values of A. Compared with headlight sight distance, it corresponds to a design speed of approximately 50 mph [80 km/h]. On high-type highways, longer curves are appropriate to improve appearance.

From the preceding discussion, it is evident that design controls for sag vertical curves differ from those for crests, and separate design values are needed. The headlight sight distance ap- pears to be the most logical criterion for general use, and the values determined for stopping sight distances are within the limits recognized in current practice. The use of this criterion to establish design values for a range of lengths of sag vertical curves is recommended. As in the case of crest vertical curves, it is convenient to express the design control in terms of the K rate for all values of A. This entails some deviation from the computed values of K for small values of A, but the differences are not significant. Table 3-37 shows the range of computed values and the rounded values of K selected as design controls. The lengths of sag vertical curves on the basis of the design speed values of K are shown by the solid lines in Figure 3-37. It is to be emphasized that these lengths are minimum values based on design speed; longer curves are desired wherever practical, but special attention to drainage should be exercised where values of K in excess of 167 ft [51 m] per percent change in grade are used.

Minimum lengths of vertical curves for flat gradients also are recognized for sag conditions. The values determined for crest conditions appear to be generally suitable for sags. Lengths of sag vertical curves, shown as vertical lines in Figure 3-37, are equal to three times the design speed in mph [0.6 times the design speed in km/h].

Sag vertical curves shorter than the lengths computed from Table 3-37 may be justified for economic reasons in cases where an existing feature, such as a structure not ready for replacement, controls the vertical profile. In certain cases, ramps may also be designed with shorter sag vertical curves. Fixed-source lighting is desirable in such cases. For street design, some engineers accept design of a sag or crest where A is about 1 percent or less without a length of calculated vertical curve. However, field modifications during construction usually result in constructing the equivalent to a vertical curve, even if short.

~~*Table 3-37. Design Controls for Sag Vertical Curves*~~

~~**U.S. Customary**~~			
~~Design Speed (mph)~~	~~Stopping Sight Distance (ft)~~	~~Rate of Vertical Curvature, K[a]~~	
		~~Calculated~~	~~Design~~
~~15~~	~~80~~	~~9.4~~	~~10~~
~~20~~	~~115~~	~~16.5~~	~~17~~
~~25~~	~~155~~	~~25.5~~	~~26~~
~~30~~	~~200~~	~~36.4~~	~~37~~
~~35~~	~~250~~	~~49.0~~	~~49~~
~~40~~	~~305~~	~~63.4~~	~~64~~
~~45~~	~~360~~	~~78.1~~	~~79~~
~~50~~	~~425~~	~~95.7~~	~~96~~
~~55~~	~~495~~	~~114.9~~	~~115~~
~~60~~	~~570~~	~~135.7~~	~~136~~
~~65~~	~~645~~	~~156.5~~	~~157~~
~~70~~	~~730~~	~~180.3~~	~~181~~
~~75~~	~~820~~	~~205.6~~	~~206~~
~~80~~	~~910~~	~~231.0~~	~~231~~

~~**Metric**~~			
~~Design Speed (km/h)~~	~~Stopping Sight Distance (m)~~	~~Rate of Vertical Curvature, K[a]~~	
		~~Calculated~~	~~Design~~
~~20~~	~~20~~	~~2.1~~	~~3~~
~~30~~	~~35~~	~~5.1~~	~~6~~
~~40~~	~~50~~	~~8.5~~	~~9~~
~~50~~	~~65~~	~~12.2~~	~~13~~
~~60~~	~~85~~	~~17.3~~	~~18~~
~~70~~	~~105~~	~~22.6~~	~~23~~
~~80~~	~~130~~	~~29.4~~	~~30~~
~~90~~	~~160~~	~~37.6~~	~~38~~
~~100~~	~~185~~	~~44.6~~	~~45~~
~~110~~	~~220~~	~~54.4~~	~~55~~
~~120~~	~~250~~	~~62.8~~	~~63~~
~~130~~	~~285~~	~~72.7~~	~~73~~

Source: *A Policy on Geometric Design of Highways and Streets* (2018) by AASHTO, Washington, D.C. Used with permission.

Proposed Table 3-37. Design Controls for Sag Vertical Curves
RURAL OR HIGH SPEED

U.S. Customary				Metric			
Design Speed (mph)	Stopping Sight Distance (ft)	Rate of Vertical Curvature, K[a]		Design Speed (km/h)	Stopping Sight Distance (m)	Rate of Vertical Curvature, K[a]	
		Calculated	Design			Calculated	Design
15	70	7.6	8	20	20	2.1	3
20	105	14.4	15	30	30	4.0	4
25	140	22.0	23	40	45	7.3	8
30	180	31.5	32	50	60	10.9	11
35	225	42.6	43	60	80	16.0	16
40	280	56.8	57	70	100	21.3	22
45	335	71.4	72	80	120	26.7	27
50	390	86.2	87	90	145	33.5	34
55	455	103.9	104	100	170	40.4	41
60	525	123.2	124	110	200	48.8	49
65	600	144.0	144	120	230	57.2	58
70	675	164.9	165	130	265	67.0	68
75	760	188.8	189				
80	845	212.7	213				

[a] Rate of vertical curvature, *K*, is the length of curve per percent algebraic difference intersecting grades (*A*), *K* = *L/A*.

LOW SPEED URBAN

U.S. Customary				Metric			
Design Speed (mph)	Stopping Sight Distance (ft)	Rate of Vertical Curvature, K[a]		Design Speed (km/h)	Stopping Sight Distance (m)	Rate of Vertical Curvature, K[a]	
		Calculated	Design			Calculated	Design
15	65	6.7	7	20	20	2.1	3
20	95	12.3	13	30	30	4.0	4
25	130	19.8	20	40	40	6.2	7
30	165	27.9	28	50	55	9.7	10
35	205	37.6	38	60	70	13.4	14
40	245	47.7	48	70	90	18.6	19
45	295	60.8	61				

[a] Rate of vertical curvature, *K*, is the length of curve per percent algebraic difference intersecting grades (*A*), *K* = *L/A*.

3.4.6.4 Sight Distance at Undercrossings

Sight distance on the highway through a grade separation should be at least as long as the minimum stopping sight distance and preferably longer. Design of the vertical alignment is the same as at any other point on the highway except in some cases of sag vertical curves underpassing a structure as illustrated in Figure 3-38. While not a frequent concern, the structure fascia may cut the line of sight and limit the sight distance to less than otherwise is attainable. It is generally practical to provide the minimum length of sag vertical curve at grade separation structures, and even where the recommended grades are exceeded, the sight distance should not need to be reduced below the minimum recommended values for stopping sight distance.

For some conditions, the designer may wish to check the available sight distance at an undercrossing, such as at a two-lane undercrossing without ramps where it would be desirable to provide passing sight distance. Such checks are best made graphically on the profile, but may be performed through computations.

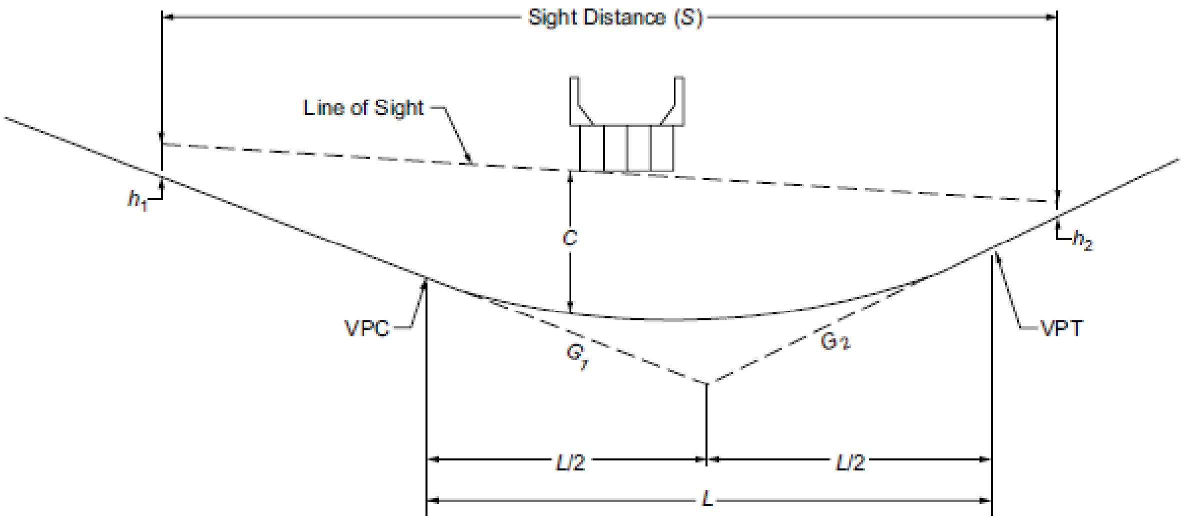


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Figure 3-38. Sight Distance at Undercrossings

The general equations for sag vertical curve length at undercrossings are:

Case 1—Sight distance greater than length of vertical curve (*S* > *L*):

U.S. Customary	Metric	
$L = 2S - \frac{800\left[C - \left(\frac{h_1 + h_2}{2}\right)\right]}{A}$	$L = 2S - \frac{800\left[C - \left(\frac{h_1 + h_2}{2}\right)\right]}{A}$	(3-53)
where: L = length of vertical curve, ft S = sight distance, ft C = vertical clearance, ft h_1 = height of eye, ft h_2 = height of object, ft A = algebraic difference in grades, percent	where: L = length of vertical curve, m S = sight distance, m C = vertical clearance, m h_1 = height of eye, m h_2 = height of object, m A = algebraic difference in grades, percent	

Case 2—Sight distance less than length of vertical curve ($S < L$):

U.S. Customary	Metric	
$L = \frac{AS^2}{800\left[C - \left(\frac{h_1 + h_2}{2}\right)\right]}$	$L = \frac{AS^2}{800\left[C - \left(\frac{h_1 + h_2}{2}\right)\right]}$	(3-54)
where: L = length of vertical curve, ft S = sight distance, ft C = vertical clearance, ft h_1 = height of eye, ft h_2 = height of object, ft	where: L = length of vertical curve, m S = sight distance, m C = vertical clearance, m h_1 = height of eye, m h_2 = height of object, m	

Using an eye height of 7.6 ft [2.3 m] ~~8.0 ft [2.4 m]~~ for a truck driver and an object height of 3.0 ft [0.9 m] ~~2.0 ft [0.6 m]~~ for the taillights of a vehicle, the following equations can be derived:

Case 1—Sight distance greater than length of vertical curve ($S > L$):

U.S. Customary	Metric	
$L = 2S - \frac{800(C - 5.3\ \cancel{5})}{A}$	$L = 2S - \frac{800(C - 1.6\ \cancel{1.5})}{A}$	(3-53)

Case 2—Sight distance less than length of vertical curve ($S < L$):

U.S. Customary	Metric	
$L = \frac{AS^2}{800(C - 5.3\ \cancel{5})}$	$L = \frac{AS^2}{800(C - 1.6\ \cancel{1.5})}$	(3-54)

5.2 Local Roads in Rural Areas

5.2.1 General Design Considerations

5.2.1.6 Cross Slope

Traveled-way cross slope should be adequate to provide proper drainage. Normally, cross slopes range from 1.5 to 2 percent for paved surfaces and 2 to 6 percent for unpaved surfaces.

For unpaved surfaces, such as stabilized or loose gravel, and for stabilized earth surfaces, a cross slope of at least 3 percent is desirable. For further information on pavement and shoulder cross slopes, see Sections 4.2.2 and 4.4.3.

Superelevation—For roads in rural areas with paved surfaces, superelevation should be not more than 12 percent, except where snow and ice conditions prevail, in which case the superelevation should be not more than 8 percent. For unpaved roads, superelevation should be not more than 12 percent.

Superelevation runoff is the length of roadway needed to accomplish a change in outside-lane cross slope from zero (flat) to full superelevation, or vice versa. Minimum lengths of runoff are presented in Section 3.3.8.2. Adjustments in design runoff lengths may be desirable for smooth riding, surface drainage, and good appearance. For a general discussion on this topic, see Section 3.3.8, "Transition Design Controls."

Sight Distance—Minimum stopping sight distance and passing sight distance should be as shown in Tables 5-3 and 5-4. The minimum SSD values in Table 5-3 assume a brake reaction time of 2.2 s and deceleration rate of 11.8 ft/s^2 [4.5 m/s^2]. Criteria for measuring sight distance, both vertical and horizontal, are as follows: for stopping sight distance, the height of eye is 3.75 ft [1.14 m] ~~3.50 ft [1.08 m]~~ and the height of object is 2.00 ft [0.60 m]; for passing sight distance, the height of eye remains the same, but the height of object is 3.75 ft [1.14 m] ~~3.50 ft [1.08 m]~~. Section 3.2 provides a general discussion of sight distance.

~~***Table 5-3. Design Controls for Stopping Sight Distance and for Crest and Sag Vertical Curves***~~

~~U.S. Customary~~			
~~Initial Speed (mph)~~	~~Design Stopping Sight Distance (ft)~~	~~Rate of Vertical Curvature, *K*[a] (ft/%)~~	
		~~Crest~~	~~Sag~~
~~15~~	~~80~~	~~3~~	~~10~~
~~20~~	~~115~~	~~7~~	~~17~~
~~25~~	~~155~~	~~12~~	~~26~~
~~30~~	~~200~~	~~19~~	~~37~~
~~35~~	~~250~~	~~29~~	~~49~~
~~40~~	~~305~~	~~44~~	~~64~~
~~45~~	~~360~~	~~61~~	~~79~~
~~50~~	~~425~~	~~84~~	~~96~~
~~55~~	~~495~~	~~114~~	~~115~~
~~60~~	~~570~~	~~151~~	~~136~~
~~65~~	~~645~~	~~193~~	~~157~~

~~Metric~~			
~~Initial Speed (km/h)~~	~~Design Stopping Sight Distance (m)~~	~~Rate of Vertical Curvature, *K*[a] (m/%)~~	
		~~Crest~~	~~Sag~~
~~20~~	~~20~~	~~1~~	~~3~~
~~30~~	~~35~~	~~2~~	~~6~~
~~40~~	~~50~~	~~4~~	~~9~~
~~50~~	~~65~~	~~7~~	~~13~~
~~60~~	~~85~~	~~11~~	~~18~~
~~70~~	~~105~~	~~17~~	~~23~~
~~80~~	~~130~~	~~26~~	~~30~~
~~90~~	~~160~~	~~39~~	~~38~~
~~100~~	~~185~~	~~52~~	~~45~~

~~[a] Rate of vertical curvature, *K*, is the length of curve per percent algebraic difference in the intersecting grades (i.e., *K* = *L*/*A*). (See Sections 3.2.2 and 3.4.6 for details.)~~

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Proposed Table 5-3. Design Controls for Stopping Sight Distance and for Crest and Sag Vertical Curves

U.S. Customary				Metric			
Initial Speed (mph)	Design Stopping Sight Distance (ft)	Rate of Vertical Curvature, K^a (ft/%)		Initial Speed (km/h)	Design Stopping Sight Distance (m)	Rate of Vertical Curvature, K^a (m/%)	
		Crest	Sag			Crest	Sag
15	70	3	8	20	20	1	3
20	105	5	15	30	30	2	4
25	140	9	23	40	45	3	8
30	180	15	32	50	60	6	11
35	225	23	43	60	80	10	16
40	280	35	57	70	100	15	22
45	335	50	72	80	120	22	27
50	390	68	87	90	145	31	34
55	455	93	104	100	170	43	41
60	525	123	124				
65	600	161	144				

[a] Rate of vertical curvature, *K*, is the length of curve per percent algebraic difference in the intersecting grades (i.e., $K = L/A$). (See Sections 3.2.2 and 3.4.6 for details.)

5.3 Local Streets in Urban Areas

5.3.1 General Design Considerations

5.3.1.8 Sight Distance

Minimum stopping sight distance for local streets should range from 65 ~~100~~ to 200 ft [20 ~~30~~ to 60 m] depending on the design speed (see Table 3-1). Design for passing sight distance seldom is applicable on local streets.

5.4 Recreational Roads

5.4.1 General Design Considerations

5.4.1.4 Vertical Alignment

Vertical curves should be comfortable for the driver, pleasing in appearance, and adequate for drainage. Minimum or greater-than-minimum stopping sight distance should be provided. The designer should consider above-minimum vertical curve lengths at driver decision points, where drainage or aesthetic problems exist, or simply to provide additional sight distance.

Vertical curve design for two-lane roads is discussed in Section 3.4.6, which also presents specific design values. Table 5-8 also includes additional information for very low design speeds not tabulated elsewhere. For two-way, single-lane roads, crest vertical curves should be significantly longer than those for two-lane roads. As previously discussed, the stopping sight distance for a two-way, single-lane road should be approximately twice the stopping sight distance for a comparable two-lane road. Table 5-8 includes K values for single-lane roads, from which vertical curve lengths can be determined. Because users of recreational roads are often unfamiliar with the area, 11.8 ft/s^2 was utilized for all stopping sight distance values in Table 5-8.

~~*Table 5-8. Design Controls for Stopping Sight Distance and for Crest and Sag Vertical Curves—Recreational Roads*~~

~~U.S. Customary~~			
~~Initial Speed (mph)~~	~~Design Stopping Sight Distance (ft)~~	~~Rate of Vertical Curvature, K^a (ft/%)~~	
		~~Crest~~	~~Sag~~
~~Two-lane roads and one-way, single-lane roads~~			
~~15~~	~~80~~	~~3~~	~~10~~
~~20~~	~~115~~	~~7~~	~~17~~
~~25~~	~~155~~	~~12~~	~~26~~
~~30~~	~~200~~	~~19~~	~~37~~
~~35~~	~~250~~	~~29~~	~~49~~
~~40~~	~~305~~	~~44~~	~~64~~
~~Two-way, single-lane roads~~			
~~15~~	~~160~~	~~12~~	~~27~~
~~20~~	~~230~~	~~25~~	~~44~~
~~25~~	~~310~~	~~45~~	~~65~~
~~30~~	~~400~~	~~74~~	~~89~~
~~35~~	~~500~~	~~116~~	~~117~~
~~40~~	~~610~~	~~172~~	~~147~~

~~Metric~~			
~~Initial Speed (km/h)~~	~~Design Stopping Sight Distance (m)~~	~~Rate of Vertical Curvature, K^a (m/%)~~	
		~~Crest~~	~~Sag~~
~~Two-lane roads and one-way, single-lane roads~~			
~~20~~	~~20~~	~~1~~	~~3~~
~~30~~	~~35~~	~~2~~	~~6~~
~~40~~	~~50~~	~~4~~	~~9~~
~~50~~	~~65~~	~~7~~	~~13~~
~~60~~	~~85~~	~~11~~	~~18~~
~~Two-way, single-lane roads~~			
~~20~~	~~40~~	~~2~~	~~6~~
~~30~~	~~70~~	~~7~~	~~13~~
~~40~~	~~100~~	~~15~~	~~21~~
~~50~~	~~130~~	~~26~~	~~29~~
~~60~~	~~170~~	~~44~~	~~40~~

Source: *A Policy on Geometric Design of Highways and Streets* (2018) by AASHTO, Washington, D.C. Used with permission.

Proposed Table 5-8. Design Controls for Stopping Sight Distance and for Crest and Sag Vertical Curves—Recreational Roads

U.S. Customary			
Initial Speed (mph)	Design Stopping Sight Distance (ft)	Rate of Vertical Curvature, K^a (ft/%)	
		Crest	Sag
Two-lane roads and one-way, single-lane roads			
15	70	3	8
20	105	5	15
25	140	9	23
30	180	15	32
35	225	23	43
40	280	35	57
Two-way, single-lane roads			
15	140	9	23
20	210	20	39
25	280	35	57
30	360	58	79
35	450	91	103
40	560	140	133

Metric			
Initial Speed (km/h)	Design Stopping Sight Distance (m)	Rate of Vertical Curvature, K^a (m/%)	
		Crest	Sag
Two-lane roads and one-way, single-lane roads			
20	20	1	3
30	30	2	4
40	45	3	8
50	60	6	11
60	80	10	16
Two-way, single-lane roads			
20	40	3	7
30	60	6	11
40	90	12	19
50	120	22	27
60	160	38	38

5.4.1.6 Sight Distance

Minimum stopping sight distance and passing sight distance are a direct function of the design speed. The subject of sight distance for two-lane roads is addressed in Section 3.2; however, sight distance design criteria are not included in Section 3.2 for roads with very low design speeds and for two-way single-lane roads. On two-way single-lane roads, sufficient sight distance should be available wherever two vehicles might approach one another so that one vehicle can reach the turnout or both vehicles can stop before colliding. Stopping sight distance should be measured using an eye height of 3.75 ft [1.14 m] ~~3.50 ft [1.08 m]~~ and a height of opposing vehicle of 4.25 ft [1.30 m]. The stopping sight distance for a two-way, single-lane road should be approximately twice the stopping sight distance that would be used in design of a comparable two-lane road. Suggested stopping sight distances for two-way, single-lane roads are given in Table 5-8. The minimum stopping sight distance values for two-lane recreational roadways displayed in Table 5-8 assume a brake reaction time of 2.2 s and deceleration rate of 11.8 ft/s^2 [4.5 m/s^2].

6.2 Collectors in Rural Areas

6.2.1 General Design Considerations

6.2.1.8 Sight Distance

Stopping sight distance and passing sight distance are a direct function of the design speed. An eye height of 3.75 ft [1.14 m] ~~3.50 ft [1.08 m]~~ and an object height of 2.0 ft [0.60 m] are used to determine stopping sight distance along with a brake reaction time of 2.2 seconds and a deceleration rate of 11.8 ft/s^2 [4.5 m/s^2]. An eye height of 3.75 ft [1.14 m] ~~3.50 ft [1.08 m]~~ and an object height of 3.75 ft [1.14 m] ~~3.50 ft [1.08 m]~~ are used to determine passing sight distance. For further information on sight distance, see Tables 6-3 and 6-4 and Section 3.2, "Sight Distance."

~~***Table 6-3. Design Controls for Stopping Sight Distance and for Crest and Sag Vertical Curves***~~

~~U.S. Customary~~				~~Metric~~			
~~Design Speed~~	~~Design Stopping Sight Distance~~	~~Rate of Vertical Curvature, K[a] (ft/%)~~		~~Design Speed~~	~~Design Stopping Sight Distance~~	~~Rate of Vertical Curvature, K[a] (m/%)~~	
~~(mph)~~	~~(ft)~~	~~crest~~	~~sag~~	~~(km/h)~~	~~(m)~~	~~Crest~~	~~Sag~~
~~20~~	~~115~~	~~7~~	~~17~~	~~30~~	~~35~~	~~2~~	~~6~~
~~25~~	~~155~~	~~12~~	~~26~~	~~40~~	~~50~~	~~4~~	~~9~~
~~30~~	~~200~~	~~19~~	~~37~~	~~50~~	~~65~~	~~7~~	~~13~~
~~35~~	~~250~~	~~29~~	~~49~~	~~60~~	~~85~~	~~11~~	~~18~~
~~40~~	~~305~~	~~44~~	~~64~~	~~70~~	~~105~~	~~17~~	~~23~~
~~45~~	~~360~~	~~61~~	~~79~~	~~80~~	~~130~~	~~26~~	~~30~~
~~50~~	~~425~~	~~84~~	~~96~~	~~90~~	~~160~~	~~39~~	~~38~~
~~55~~	~~495~~	~~114~~	~~115~~	~~100~~	~~185~~	~~52~~	~~45~~
~~60~~	~~570~~	~~151~~	~~136~~				
~~65~~	~~645~~	~~193~~	~~157~~				

~~[a] Rate of vertical curvature, *K*, is the length of curve per percent algebraic difference in the intersecting grades (i.e., *K* = *L*/*A*). (See Sections 3.2.2 and 3.4.6 for details.)~~

Source: *A Policy on Geometric Design of Highways and Streets* (2018) by AASHTO, Washington, D.C. Used with permission.

Proposed Table 6-3. Design Controls for Stopping Sight Distance and for Crest and Sag Vertical Curves

U.S. Customary				Metric			
Design Speed	Design Stopping Sight Distance	Rate of Vertical Curvature, K^a(ft/%)		Design Speed	Design Stopping Sight Distance	Rate of Vertical Curvature, K^a (m/%)	
(mph)	(ft)	crest	sag	(km/h)	(m)	Crest	Sag
20	105	5	15	30	30	2	4
25	140	9	23	40	45	3	8
30	180	15	32	50	60	6	11
35	225	23	43	60	80	10	16
40	280	35	57	70	100	15	22
45	335	50	72	80	120	22	27
50	390	68	87	90	145	31	34
55	455	93	104	100	170	43	41
60	525	123	124				
65	600	161	144				

[a]Rate of vertical curvature, *K*, is the length of curve per percent algebraic difference in the intersecting grades (i.e., $K = L/A$). (See Sections 3.2.2 and 3.4.6 for details.)

6.3 Collectors in Urban Areas

6.3.1 General Design Considerations

6.3.1.8 Sight Distance

Stopping sight distance for collector streets, including the assumptions for the recommended deceleration rate, varies with design speed. A deceleration rate of 15.0 ft/s^2 [4.5 m/s^2] may be utilized to determine stopping sight distance on urban collectors with speeds less than or equal to 45 mph. However, for urban collectors with speeds greater than 45 mph, a deceleration rate of 11.8 ft/s^2 [3.6 m/s^2] is recommended. A brake reaction time of 2.2 s is recommended across all speeds. Design for passing sight distance is seldom appropriate on collector streets. For further information, see Tables 6-3 and 6-4, as well as Section 3.2, “Sight Distance.”

7.2 Arterials in Rural Areas

7.2.2 General Design Considerations

7.2.2.4 Sight Distance

Sight distance is directly related to and varies appreciably with design speed. Stopping sight distance should be provided throughout the length of the roadway. Passing and decision sight distances influence roadway operations and should be provided wherever practical. Providing decision sight distance at locations where complex decisions are made greatly enhances the capability for drivers to accomplish maneuvers. Examples of locations where complex decisions are needed include interchanges, high-volume intersections, transitions in roadway width, and transitions in the number of lanes. Providing adequate sight distance on arterials in rural areas, which may combine both high speeds and high traffic volumes, can be complex. Table 7-1 presents the recommended minimum values of stopping and passing sight distance. The stopping sight distance values provided in Table 7-1 assume a brake reaction time of 2.2 s and a deceleration rate of 11.8 ft/s^2 [4.5 m/s^2]. Refer to Section 3.2 for a comprehensive discussion of sight distance and for tabulated values for decision sight distance.

~~Table 7-1. Minimum Sight Distances for Arterials in Rural Areas~~

~~U.S. Customary~~			~~Metric~~		
~~Design Speed (mph)~~	~~Minimum Stopping Sight Distance (ft)~~	~~Minimum Passing Sight Distance (ft)~~	~~Design Speed (km/h)~~	~~Minimum Stopping Sight Distance (m)~~	~~Minimum Passing Sight Distance (m)~~
~~20~~	~~115~~	~~400~~	~~30~~	~~35~~	~~120~~
~~25~~	~~155~~	~~450~~	~~40~~	~~50~~	~~140~~
~~30~~	~~200~~	~~500~~	~~50~~	~~65~~	~~160~~
~~35~~	~~250~~	~~550~~	~~60~~	~~85~~	~~180~~
~~40~~	~~305~~	~~600~~	~~70~~	~~105~~	~~210~~
~~45~~	~~360~~	~~700~~	~~80~~	~~130~~	~~245~~
~~50~~	~~425~~	~~800~~	~~90~~	~~160~~	~~280~~
~~55~~	~~495~~	~~900~~	~~100~~	~~185~~	~~320~~
~~60~~	~~570~~	~~1000~~	~~110~~	~~220~~	~~355~~
~~65~~	~~645~~	~~1100~~	~~120~~	~~250~~	~~395~~
~~70~~	~~730~~	~~1200~~	~~130~~	~~285~~	~~440~~
~~75~~	~~820~~	~~1300~~			
~~80~~	~~910~~	~~1400~~			

Source: *A Policy on Geometric Design of Highways and Streets* (2018) by AASHTO, Washington, D.C. Used with permission.

Proposed Table 7-1. Minimum Sight Distances for Arterials in Rural Areas

U.S. Customary			Metric		
Design Speed (mph)	Minimum Stopping Sight Distance (ft)	Minimum Passing Sight Distance (ft)	Design Speed (km/h)	Minimum Stopping Sight Distance (m)	Minimum Passing Sight Distance (m)
20	105	400	30	30	120
25	140	450	40	45	140
30	180	500	50	60	160
35	225	550	60	80	180
40	280	600	70	100	210
45	335	700	80	120	245
50	390	800	90	145	280
55	455	900	100	170	320
60	525	1000	110	200	355
65	600	1100	120	230	395
70	675	1200	130	265	440
75	760	1300			
80	845	1400			

Ideally, intersections and railroad crossings should be grade separated or provided with adequate sight distance. Intersections should be placed in sag or tangent locations, where practical, to provide maximum visibility of the roadway, signs, and pavement markings.

7.3 Arterials in Urban Areas

7.3.2 General Design Considerations

7.3.2.4 Sight Distance

Providing adequate sight distance is important in the design of arterials in urban areas. Sight distance affects normal operational characteristics, particularly where roadways carry high traffic volumes, and are important to the visibility of pedestrians and bicyclists as well. The sight distance values given in Table 7-1 are also applicable to the design of arterials in urban areas when speeds are greater than 45 mph. For urban arterials with speeds less than or equal to 45 mph, SSD should be computed utilizing a brake reaction time of 2.2 s and deceleration rate of 15 ft/s^2. Design values for intersection sight distance are presented in Section 9.5.

9.5 Intersection Sight Distance

9.5.2 Sight Triangles

9.5.2.3 Identification of Sight Obstructions within Sight Triangles

The profiles of the intersecting roadways should be designed to provide the recommended sight distances for drivers on the intersection approaches. Within a sight triangle, any object at a height above the elevation of the adjacent roadways that would obstruct the driver's view should be removed or lowered, if practical. Such objects may include buildings, parked vehicles, roadway structures, roadside hardware, hedges, trees, bushes, unmowed vegetation, tall crops, walls, fences, and the terrain itself. Particular attention should be given to the evaluation of clear sight triangles at interchange ramp/crossroad intersections where features such as bridge railings, roadside barriers, piers, and abutments are potential sight obstructions.

The determination of whether an object constitutes a sight obstruction should consider both the horizontal and vertical alignment of both intersecting roadways, as well as the height and position of the object. In making this determination, it should be assumed that the driver's eye is 3.75 ft [1.14 m] ~~3.5 ft [1.08 m]~~ above the roadway surface and that the object to be seen is 3.75 ft [1.14 m] ~~3.5 ft [1.08 m]~~ above the surface of the intersecting road.

This object height ~~is based on a vehicle height of 4.35 ft [1.33 m], which represents the 15th percentile of vehicle heights in the current passenger car population less an allowance of 10 in. [250 mm]. This allowance~~ represents a near-maximum value for the portion of a passenger car height that needs to be visible for another driver to recognize it as the object. The use of an object height equal to the driver eye height makes intersection sight distances reciprocal (i.e., if one driver can see another vehicle, then the driver of that vehicle can also see the first vehicle).

Where the sight-distance value used in design is based on a single-unit or combination truck as the design vehicle, it is also appropriate to use the eye height of a truck driver in checking sight obstructions. The recommended value of a truck driver's eye height is 7.6 ft [2.33 m] above the roadway surface.

9.6 Turning Roadways and Channelization

9.6.5 Stopping Sight Distance at Intersections for Turning Roadways

9.6.5.1 General Considerations

The values for stopping sight distance as computed in Section 3.2.2 for open highway conditions are applicable to turning roadway intersections of the same design speed. The values from Section 3.2.2, together with the value for a design speed in increments of 5 mph [10 km/h]), are shown in Table 9-19. These values were computed assuming a brake reaction time of 2.2 s and a deceleration rate of 11.8 ft/s^2 [4.5 m/s^2].

~~*Table 9-19. Stopping Sight Distance for Turning Roadways*~~

~~U.S. Customary~~								
~~Design speed (mph)~~	~~10~~	~~15~~	~~20~~	~~25~~	~~30~~	~~35~~	~~40~~	~~45~~
~~Stopping sight distance (ft)~~	~~50~~	~~80~~	~~115~~	~~155~~	~~200~~	~~250~~	~~305~~	~~360~~

~~Metric~~							
~~Design speed (km/h)~~	~~15~~	~~20~~	~~30~~	~~40~~	~~50~~	~~60~~	~~70~~
~~Stopping sight distance (m)~~	~~15~~	~~20~~	~~35~~	~~50~~	~~65~~	~~85~~	~~105~~

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Proposed Table 9-19. Stopping Sight Distance for Turning Roadways

U.S. Customary								
Design speed (mph)	10	15	20	25	30	35	40	45
Stopping sight distance (ft)	45	70	105	140	180	225	280	335

Metric							
Design speed (km/h)	10	20	30	40	50	60	70
Stopping sight distance (m)	10	20	30	45	60	80	100

These sight distances should be available at all points along a turning roadway; wherever practical, longer sight distances should be provided. They apply as controls in design of both vertical and horizontal alignment.

9.6.5.2 Vertical Control

The length of vertical curve is predicated, as it is for open highway conditions, on sight distance measured from the height of eye of 3.75 ft [1.14 m] ~~3.5 ft [1.08 m]~~ to the height of object of 2 ft [0.60 m]. Equations shown in Section 3.4.6.2 apply directly.

For design speeds of less than 40 mph [60 km/h], sag vertical curves, as governed by headlight sight distances, theoretically should be longer than crest vertical curves. Lengths of sag vertical curves are found by substituting the stopping sight distances from Table 9-19 in the formulas in Section 3.4.6.3. Because the design speed of most turning roadways is governed by the horizontal curvature and the curvature is relatively sharp, a headlight beam parallel to the longitudinal axis of the vehicle ceases to be a control. Where practical, longer lengths for both crest and sag vertical curves should be used.

9.7 AUXILIARY LANES

9.7.2 Deceleration Lanes

9.7.2.1 Perception–Reaction Distance

The perception–reaction distance (d_1) in Figure 9-32 represents the distance traveled while a driver recognizes the upcoming turn lane and prepares for the left-turn maneuver. The distance increases with perception–reaction time and speed. The perception–reaction time varies with the driver's familiarity with the roadway segment and state of alertness; for example, an alert driver who is familiar with the roadway and traffic conditions has a smaller perception–reaction time than an unfamiliar driver. Traffic conditions on urban and suburban roadways could result in drivers having a higher level of alertness than those on highways in rural areas. Therefore, a value of 1.5 s is often used as the perception–reaction time for suburban, urban, urban core, and rural town contexts, and 2.5 s is often used for rural contexts (*44*).

Provision for deceleration clear of the through-traffic lanes is a desirable objective on arterial roads and streets and should be incorporated into design, whenever practical. Approximately two-thirds of the drivers observed making left turns in a research study concerning turn lanes used deceleration rates greater than 6.5 ft/s2 [2.0 m/s^2] to come to a stop at the stop line (*16*). A turn lane design based on that rate will accommodate the preferred behavior of 85 percent of turning drivers at high-speed sites. Table 9-20 presents the estimated distances needed by drivers to maneuver from the through lane into a left- or right-turn lane and brake to a stop based on an equivalent deceleration rate of 6.5 ft/s^2 [2.0 m/s^2]. These distances are based on accommodating observed driver behavior; drivers and vehicles are capable of much greater comfortable, controlled deceleration, when needed. Since provision of deceleration length based deceleration at a rate of 6.5 ft/s^2 [2.0 m/s^2] is not always practical, it should be noted that drivers are capable of much higher deceleration rates. For example, the stopping sight distance calculations in Chapter 3 ~~use 11.2 ft/s² [3.4 m/s²] as a~~ assume a higher threshold for comfortable, controlled deceleration. ~~threshold for most drivers and~~ Further, the Access Management Manual (*48*) presents distances for "limiting conditions" based on the equivalent of a 9.9-ft/s^2 [3.0-m/s^2] deceleration rate throughout the full deceleration length (i.e., taper and full-width deceleration lane). Thus, deceleration rates greater than 6.5 ft/s^2 [2.0 m/s^2] may be used where needed.

As noted above, it is not practical on many facilities to provide the full length of the auxiliary lane for deceleration due to constraints such as restricted right-of-way, distance available between adjacent intersections, and storage needs. However, research has demonstrated that providing a left- and right-turn lane on any intersection approach has a substantial crash reduction benefit (22). Therefore, turn lanes should be installed where warranted (see Section 9.7.3), even where the distances in Table 9-20 cannot be achieved.

9.12 Railroad-Highway Grade Crossings

9.12.4 Sight Distance

Sight distance is a primary consideration at crossings without train-activated warning devices. A complete discussion of sight distance at grade crossings can be found in *Railroad–Highway Grade Crossing Surfaces* (24) and NCHRP Report 288 (45).

As in the case of a roadway intersection, there are several events that can occur at a railroad– highway grade intersection without train-activated warning devices. Two of these events related to determining the sight distance are:

- The vehicle operator can observe the approaching train in a sight line that will allow the vehicle to pass through the grade crossing prior to the train's arrival at the crossing.
- The vehicle operator can observe the approaching train in a sight line that will permit the vehicle to be brought to a stop prior to encroachment in the crossing area.

Both of these maneuvers are shown as Case A illustrated in Figure 9-67. The sight triangle consists of the two major legs (i.e., the sight distance, d_H, along the roadway and the sight distance, d_T, along the railroad tracks). Values of the sight distances for various speeds of the vehicle and the train are developed from two basic equations:

U.S. Customary	Metric

$$d_H = AV_v t + \frac{BV_v^2}{a} + D + d_e$$

$$d_T = \frac{V_T}{V_V}\left[(A)V_v t + \frac{BV_v^2}{a} + 2D + L + W\right]$$

where:

A = constant = 1.47

B = constant = 1.075

d_H = sight-distance leg along the highway allows a vehicle proceeding to speed V_v to cross tracks even though a train is observed at a distance d_T from the crossing or to stop the vehicle without encroachment of the crossing area (ft)

d_T = sight-distance leg along the railroad tracks to permit the maneuvers described as for d_H (ft)

V_v = speed of the vehicle (mph)

V_T = speed of the train (mph)

t = perception/reaction time, which is assumed to be ~~2.5~~ 2.2 s (This is the same value used in Section 3.1 to determine the stopping sight distance.)

a = driver deceleration, which is assumed to be ~~11.2~~ 11.8 ft/s² (This is the same value used in Section 3.1 to determine stopping sight distance.)

D = distance from the stop line or front of the vehicle to the nearest rail, which is assumed to be 15 ft

d_e = distance from the driver to the front of the vehicle, which is assumed to be 8 ft

L = length of vehicle, which is assumed to be 73.5 ft

W = distance between outer rails (for a single track, this value is 5 ft)

$$d_H = AV_v t + \frac{BV_v^2}{a} + D + d_e \qquad (9\text{-}5)$$

$$d_T = \frac{V_T}{V_V}\left[(A)V_v t + \frac{BV_v^2}{a} + 2D + L + W\right]$$

where:

A = constant = 0.278

B = constant = 0.039

d_H = sight-distance leg along the roadway allows a vehicle proceeding to speed V_v to cross tracks even though a train is observed at a distance d_T from the crossing or to stop the vehicle without encroachment of the crossing area (m)

d_T = sight-distance leg along the railroad tracks to permit the maneuvers described as for d_H (m)

V_v = speed of the vehicle (km/h)

V_T = speed of the train (km/h)

t = perception/reaction time, which is assumed to be ~~2.5~~ 2.2 s (This is the same value used in Section 3.1 to determine the stopping sight distance.)

a = driver deceleration, which is assumed to be ~~3.4~~ 3.6 m/s² (This is the same value used in Section 3.1 to determine stopping sight distance.)

D = distance from the stop line or front of the vehicle to the nearest rail, which is assumed to be 4.5 m

d_e = distance from the driver to the front of the vehicle, which is assumed to be 2.4 m

L = length of vehicle, which is assumed to be 22.4 m

W = distance between outer rails (for a single track, this value is 1.5 m)

Note: Adjustments should be made for skewed crossings and roadway grades that are other than flat

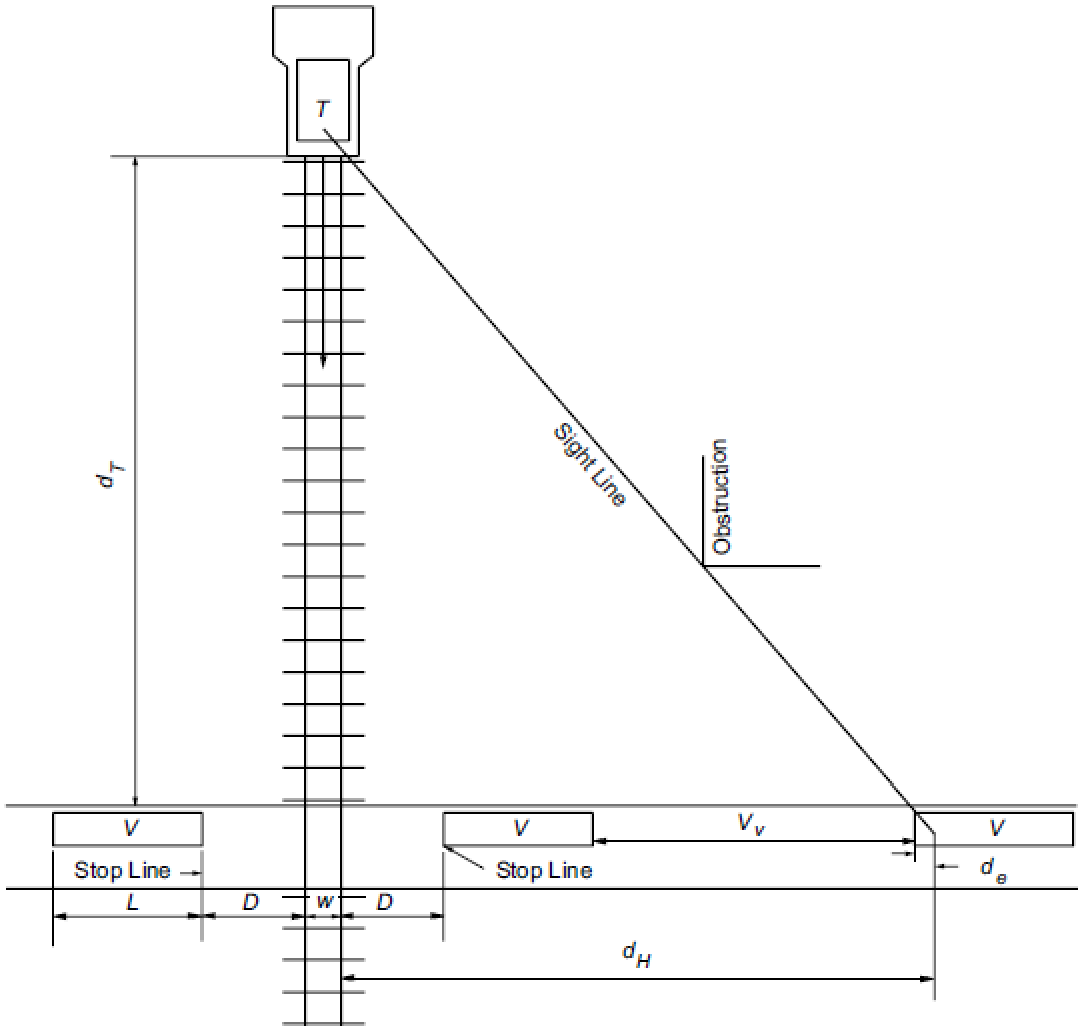

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Figure 9-67. Case A: Moving Vehicle to Cross or Stop at Railroad Crossing

The values for Case B illustrated in Figure 9-68 represent departure sight distance for a range of train speeds. When a vehicle has stopped at a railroad crossing, the next maneuver is to depart from the stopped position. The vehicle operator should have sufficient sight distance along the tracks to accelerate the vehicle and clear the crossing prior to the arrival of a train, even if the train comes into view just as the vehicle starts, as shown in Figure 9-68. These values are obtained from the equation:

U.S. Customary	**Metric**	
$d_T = AV_T\left[\frac{V_G}{a_1} + \frac{L + 2D + W - d_a}{V_G} + J\right]$	$d_T = AV_T\left[\frac{V_G}{a_1} + \frac{L + 2D + W - d_a}{V_G} + J\right]$	(9-6)
where:	where:	
d_T= sight distance leg along the railroad tracks	d_T= sight distance leg along the railroad	

for the departure maneuver (ft)

A = constant = 1.47

d_H = sight-distance leg along the highway allows a vehicle proceeding to speed V_v to cross tracks even though a train is observed at a distance d_T from the crossing or to stop the vehicle without encroachment of the crossing area (ft)

d_T = sight-distance leg along the railroad tracks to permit the maneuvers described as for d_H (ft)

V_T = speed of the train (mph)

V_G = maximum speed of vehicle in first gear, which is assumed to be 8.8 ft/s

a_1 = acceleration of vehicle in first gear, which is assumed to be 1.47 ft/s^2

L = length of vehicle, which is assumed to be 73.5 ft

D = distance from the stop line or front of the vehicle to the nearest rail, which is assumed to be 15 ft

J = sum of perception and time to activate clutch or automatic shift, which is assumed to be 2.0 s

W = distance between outer rails (for a single track, this value is 5 ft)

$$d_a = \frac{V_G^2}{2a_1}$$

where:

d_a = distance vehicle travels while accelerating to maximum speed in first gear (ft)

$$d_a = \frac{V_G^2}{2a_1} = \frac{8.8^2}{(2)(1.47)} = 26.3\ ft$$

tracks for the departure maneuver (m)

A = constant = 0.278

d_H = sight-distance leg along the roadway allows a vehicle proceeding to speed V_v to cross tracks even though a train is observed at a distance d_T from the crossing or to stop the vehicle without encroachment of the crossing area (m)

d_T = sight-distance leg along the railroad tracks to permit the maneuvers described as for d_H (m)

V_T = speed of the train (km/h)

V_G = maximum speed of vehicle in first gear, which is assumed to be 2.7 m/s

a_1 = acceleration of vehicle in first gear, which is assumed to be 0.45 m/s^2

L = length of vehicle, which is assumed to be 22.4 m

D = distance from the stop line or front of the vehicle to the nearest rail, which is assumed to be 4.5 m

J = sum of perception and time to activate clutch or automatic shift, which is assumed to be 2.0 s

W = distance between outer rails (for a single track, this value is 1.5 m)

$$d_a = \frac{V_G^2}{2a_1}$$

where:

d_a = distance vehicle travels while accelerating to maximum speed in first gear (m)

$$d_a = \frac{V_G^2}{2a_1} = \frac{2.7^2}{(2)(0.45)} = 8.1\ m$$

Note: Adjustments should be made for skewed crossings and roadway grades that are other than flat

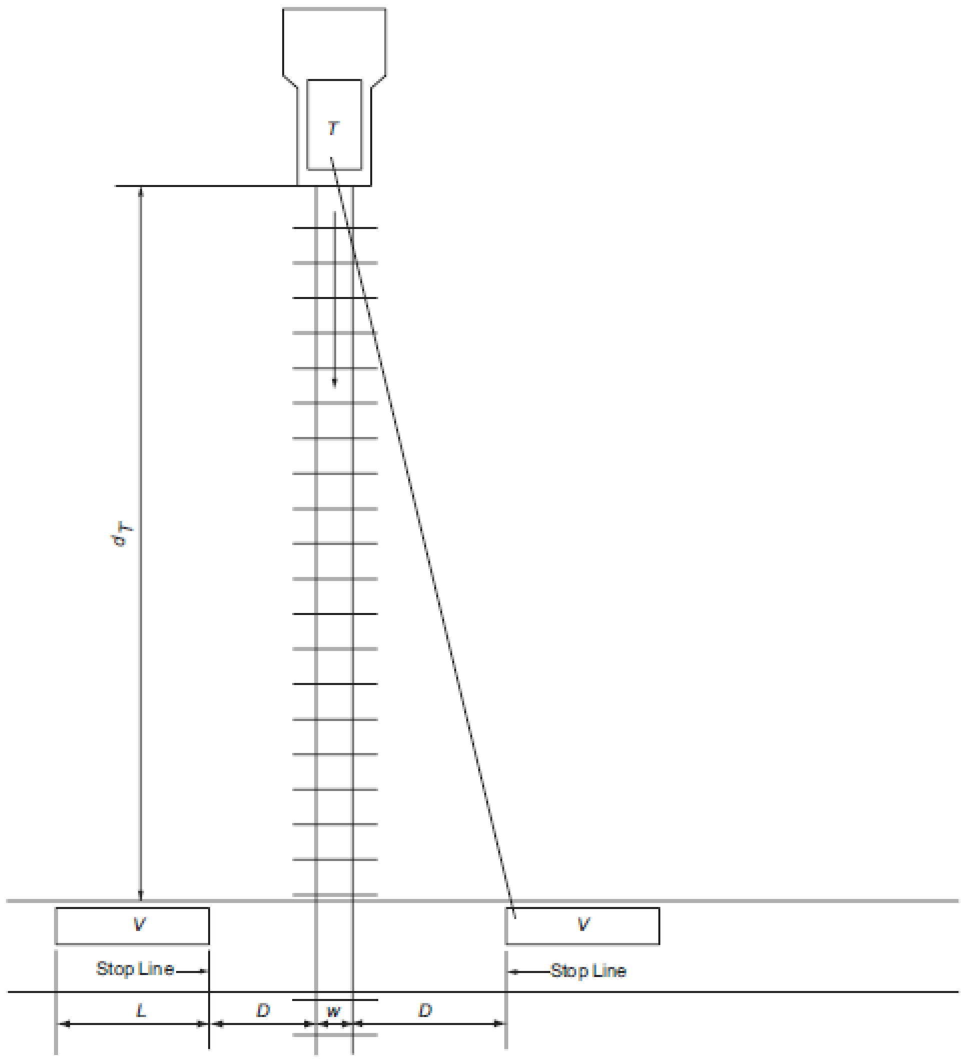

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Figure 9-68. Case B: Departure of Vehicle from Stopped Position to Cross Single Railroad Track

Table 9-29 indicates the values of the sight distances for various speeds of the vehicle and the train for Case A as determined by Equation 9-5 and the departure sight distance for a range of train speeds for Case B as determined by Equation 9-6. Sight distances of the order shown in Table 9-29 are desirable at any railroad grade crossing not controlled by active warning devices. Their attainment, however, is difficult and often impractical, except in flat, open terrain.

~~*Table 9-29. Design Sight Distance for Combination of Motor Vehicle and Train Speeds; 73.5-ft [22.4-m] Truck Crossing a Single Set of Tracks at 90 Degrees*~~

U.S. Customary									
Train Speed (mph)	Case B Departure from Stop (ft)	Case A Moving Vehicle							
		Vehicle Speed (mph)							
	0	10	20	30	40	50	60	70	80
		Distance along railroad from crossing, d_T (ft)							
10	255	155	110	102	102	106	112	119	127
20	509	310	220	203	205	213	225	239	254
30	794	465	331	305	307	319	337	358	381
40	1019	619	441	407	409	426	450	478	508
50	1273	774	551	509	511	532	562	597	635
60	1528	929	661	610	614	639	675	717	763
70	1783	1084	771	712	716	745	787	836	890
80	2037	1239	882	814	818	852	899	956	1017
90	2292	1394	992	915	920	958	1012	1075	1144
		Distance along roadway from Crossing, d_H (ft)							
		69	135	220	324	447	589	751	931

Metric														
Train Speed (km/h)	Case B Departure from Stop (m)	Case A Moving Vehicle												
		Vehicle Speed (km/h)												
	0	10	20	30	40	50	60	70	80	90	100	110	120	130
		Distance along railroad from crossing, d_T (m)												
20	96	82	51	43	40	39	39	39	40	42	43	45	47	49
40	191	164	103	85	79	77	77	79	81	84	87	90	94	98
60	287	246	154	128	119	116	116	118	121	126	130	135	141	146
80	382	328	206	171	158	154	155	157	162	167	174	180	188	195
100	478	410	257	214	198	193	193	197	202	209	217	226	235	244
120	573	492	308	256	237	231	232	236	243	251	261	271	281	293
140	669	574	360	299	277	270	270	276	283	293	304	316	328	341
		Distance along roadway from Crossing, d_H (m)												
		15	25	38	53	70	90	112	136	162	191	222	255	291

Source: *A Policy on Geometric Design of Highways and Streets* (2018) by AASHTO, Washington, D.C. Used with permission.

Proposed Table 9-29. Design Sight Distance for Combination of Motor Vehicle and Train Speeds; 73.5-ft [22.4-m] Truck Crossing a Single Set of Tracks at 90 Degrees

U.S. Customary									
Train Speed (mph)	Case B Departure from Stop (ft)	Case A Moving Vehicle Vehicle Speed (mph)							
	0	10	20	30	40	50	60	70	80
		Distance along railroad from crossing, d_T (ft)							
10	255	150	105	96	96	100	105	112	119
20	509	300	210	192	192	199	210	223	238
30	794	450	314	288	288	299	315	335	356
40	1019	600	419	383	384	398	420	446	475
50	1273	750	524	479	480	498	525	558	594
60	1528	900	629	575	575	598	631	670	713
70	1783	1050	734	671	671	697	736	781	831
80	2037	1200	838	767	767	797	841	893	950
90	2292	1350	943	863	863	896	946	1005	1069
		Distance along roadway from Crossing, d_H (ft)							
		64	124	202	298	412	545	696	865

Metric														
Train Speed (km/h)	Case B Departure from Stop (m)	Case A Moving Vehicle Vehicle Speed (km/h)												
	0	10	20	30	40	50	60	70	80	90	100	110	120	130
		Distance along railroad from crossing, d_T (m)												
20	96	80	49	41	37	36	36	37	38	39	40	42	44	45
40	191	160	99	81	75	72	72	74	76	78	81	84	87	91
60	287	241	148	122	112	109	109	110	113	117	121	126	131	136
80	382	321	198	163	149	145	145	147	151	156	162	168	175	182
100	478	401	247	203	187	181	181	184	189	195	202	210	219	227
120	573	481	297	244	224	217	217	221	227	234	243	252	262	273
140	669	561	346	285	261	254	253	258	265	273	283	294	306	318
		Distance along roadway from Crossing, d_H (m)												
		14	23	35	49	65	83	103	125	150	176	205	236	269

In other than flat terrain, it may be appropriate to rely on speed control signs and devices and to predicate sight distance on a reduced vehicle speed of operation. Where sight obstructions are present, it may be appropriate to install active traffic control devices that will bring all roadway traffic to a stop before crossing the tracks and will warn drivers automatically in time for an approaching train.

The driver of a stopped vehicle at a crossing should see enough of the railroad track to be able to cross it before a train reaches the crossing, even though the train may come into view immediately after the vehicle starts to cross. The length of the railroad track in view on each side of the crossing should be greater than the product of the train speed and the time needed for the stopped vehicle to start and cross the railroad. The sight distance along the railroad track may be determined in the same manner as it is for a stopped vehicle on a minor road to cross a major road, which is covered in Section 9.5. In order for vehicles to cross two tracks from a stopped position, with the front of the vehicle 15 ft [4.5 m] from the closest rail, sight distances along the railroad, in feet [meters], should be determined by the formula with a proper adjustment for the *W* value.

The roadway traveled way at a railroad crossing should be constructed for a suitable length with all-weather surfacing. A roadway section equivalent to the current or proposed cross section of the approach roadway should be carried across the crossing. The crossing surface itself should have a riding quality equivalent to that of the approach roadway. If the crossing surface is in poor condition, the driver's attention may be devoted to choosing the smoothest path over the crossing. This effort may well reduce the attention given to observance of the warning devices or even the approaching train. Information concerning various surface types that may be used can be found in *Railroad-Highway Grade Crossing Surfaces* (*24*).

10.9 Interchanges

10.9.6 Ramps

10.9.6.4 Ramp Terminals

10.9.6.4.7 Taper-Type Entrances

Drivers leaving a highway at an interchange are required to reduce speed as they exit onto a ramp. Drivers entering a highway from a turning roadway accelerate until the desired highway speed is reached. Because the change in speed is usually substantial, provision should be made for acceleration and deceleration to be accomplished on auxiliary lanes to minimize interference with through traffic and to reduce crash potential. Such an auxiliary lane, including tapered areas, may be referred to as a speed-change lane. The terms "speed-change lane," "deceleration lane," or "acceleration lane" as used herein apply broadly to the added lane that joins the traveled way of the highway to the turning roadway and do not necessarily imply a definite lane of uniform width. This additional lane is a part of the elongated ramp terminal area.

A speed-change lane should have sufficient length to enable a driver to make the appropriate change in speed between the highway and the turning roadway. Crashes have been shown to generally decline as the length of the acceleration or deceleration lane is increased (*NCHRP 15-75*). These results tend to vary based upon the configuration of the speed-change lane. Moreover, in the case of an acceleration lane, there should be additional length to permit adjustments in speeds of both through and entering vehicles so that the entering driver can position the vehicle opposite a gap in the through-traffic stream and then maneuver into the stream before the acceleration lane ends. This is a particular concern where the acceleration lane is preceded by a loop entry ramp, where crash risks are more pronounced (*NCHRP 15-75*). This latter consideration also influences both the configuration and length of an acceleration lane.

Two general forms of speed-change lanes are (1) the taper type and (2) the parallel type. The taper type provides a direct entry or exit at a flat angle, whereas the parallel type has an added lane for changing speed. Either type, when properly designed, will operate satisfactorily. However, the parallel type is still favored in certain areas, and some agencies use the taper type for exits and the parallel type for entrances. Furthermore, taper type entrances have been found to encourage merge speeds that are closer to freeway speeds than parallel type entrances (21), however; where there are main-line volumes that meet or exceed capacity, parallel type entrances allow additional flexibility to drivers in selecting a merge location.

10.9.6.5 Single-Lane Free-Flow Terminals, Entrances

10.9.6.5.1 Taper-Type Entrances

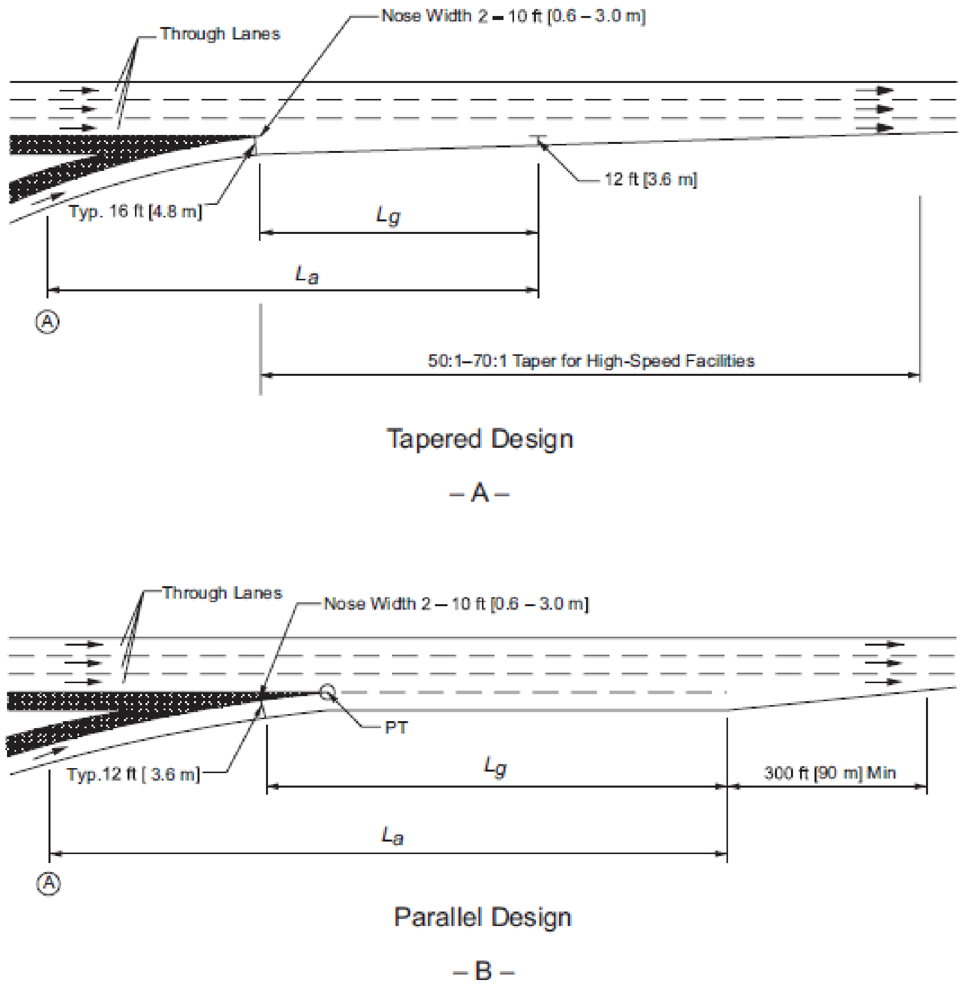
When properly designed, the taper-type entrance usually operates smoothly at all volumes up to and including the design capacity of merging areas. By relatively minor speed adjustment, the entering driver can see and use an available gap in the through-traffic stream. A typical single-lane, taper-type entrance terminal is shown in Figure 10-72A.

The entrance is merged into the freeway with a long, uniform taper. Operational studies show a desirable rate of taper of approximately 50:1 to 70:1 (longitudinal to lateral) between the outer edge of the acceleration lane and the edge of the through-traffic lane. The gap acceptance length, *Lg*, is also a consideration in the design of taper-type entrances, as illustrated in Figure 10-72A.

The geometrics of the ramp proper should be such that motorists may attain a merge speed that is within 5 mph [10 km/h] of the operating speed of the freeway by the time they reach the point where the left edge of the ramp joins the traveled way of the freeway. For consistency of application, this point of convergence of the left edge of the ramp and the right edge of the through lane may be assumed to occur where the right edge of the ramp traveled way is 12 ft [3.6 m] from the right edge of the through lane of the freeway. While it is desirable for motorists to merge onto the freeway at speeds near the operating speed of the freeway, some motorists may choose to enter the freeway at speeds below the operating speed of the freeway without using the full length of the speed-change lane. Taper type entrances have been shown to encourage motorists to merge closer to freeway speeds (21).

The distance needed for acceleration in advance of this point of convergence is governed by the speed differential between the ~~operating~~ design speed (or advisory speed) on the controlling feature (e.g., horizontal curve) of the ramp and the highway. The use of curve design speed is a departure from prior AASHTO guidance, which utilized assumed values for average running speed at the controlling curve. Recent field studies showed that passenger vehicle speeds exiting the controlling horizontal curve tended to exceed the prior AASHTO assumed values for average running speeds at the curve. In the case of a straight ramp, the controlling feature is the crossroad ramp terminal, and in the case of a loop ramp, the controlling feature is the entrance curve to the acceleration lane. At crossroad terminals where many vehicles do not begin accelerating from a stopped position, it is reasonable to assume initial speeds higher than zero when determining minimum acceleration lengths. Table 10-4 shows minimum lengths of acceleration distances for entrance terminals. Figure 10-72 shows the minimum lengths for gap acceptance. Referring to Figure 10-72, the larger value of the acceleration length (L_a) or the gap acceptance (L_g) length is suggested for use in the design of the ramp entrance. Where the minimum values for nose width (2 ft [0.6 m]), lane width 16 ft [4.8 m]), and taper rate (50:1) are used with high traffic volumes, taper lengths longer than the larger of *La* or *Lg* may be needed to avoid inferior operation and to reduce abrupt moves when merging into the main-line traffic stream. Where grades are present on ramps, speed-change lengths should be adjusted in accordance with Table 10-5.

The design values in Table 10-4 are conservative estimates based on free-merge conditions (i.e., free-flow conditions) for passenger cars. Additionally, if trucks constitute a substantial percentage of the traffic volume to be selected as the design vehicle, acceleration lane lengths designed to better accommodate heavier design vehicles can be derived using Figures 3-24 and 3-25 in Chapter 3.



Notes:

1. L_a is the recommended acceleration length as shown in Table 10-4 or as adjusted by Table 10-5.
2. Point A controls speed on the ramp. L_a should not start back on the curvature of the ramp unless the radius equals 1,000 ft [300 m] or more.
3. L_g is the recommended gap acceptance length. L_g should be a minimum of 300 to 500 ft [90 to 150 m] depending on the nose width.
4. The value of L_a or L_g, whichever produces the greater distance downstream from where the nose equals 2 ft [0.6 m], is suggested for use in the design of the ramp distance.

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Figure 10-72. Typical Single-Lane Entrance Ramps

10.9.6.5.2 Parallel-Type Entrances

The parallel-type entrance provides an added lane of sufficient length to enable a vehicle to accelerate to near-freeway speed prior to merging. A taper is provided at the end of the added lane. The process of entering the freeway is similar to a lane change to the left. The driver is able to use the side-view and rear-view mirrors to monitor surrounding traffic.

A typical design of a parallel-type entrance is shown in Figure 10-72B. Desirably, a curve with a radius of 1,000 ft [300 m] or more and a length of at least 200 ft [60 m] should be provided in advance of the added lane. If this curve has a short radius, motorists tend to drive directly onto the freeway without using the acceleration lane. This behavior results in undesirable merging operations.

The taper at the downstream end of a parallel-type acceleration lane should be a suitable length to guide the vehicle gradually onto the through lane of the freeway. A taper length of approximately 300 ft [90 m] is suitable for design speeds up to 70 mph [110 km/h].

The length of a parallel-type acceleration lane is generally measured from the point where the left edge of the traveled way of the ramp joins the traveled way of the freeway to the beginning of the downstream taper. Whereas, in the case of the taper-type entrance, acceleration is accomplished on the ramp upstream from the point of convergence of the two roadways; acceleration usually takes place downstream from this point in the case of the parallel-type entrance. However, a part of the ramp proper may also be considered in the acceleration length, provided the curve approaching the acceleration lane has a long radius of approximately 1,000 ft [300 m] or more and the motorist on the ramp has an unobstructed view of traffic on the freeway to the motorist's left. The minimum acceleration lengths for entrance terminals are given in Table 10-4, and the adjustments for grades are given in Table 10-5.

The advantages in efficient traffic operations and low crash frequencies of long acceleration lanes provided by parallel type entrances are well recognized. A long acceleration lane provides more time for the merging vehicles to find an opening in the through-traffic stream. An acceleration lane length of at least 1,200 ft [360 m] plus the taper is desirable wherever it is anticipated that the ramp and freeway will frequently carry traffic volumes approximately equal to the design capacity of the merging area.

~~Table 10-4. Minimum Acceleration Lane Lengths for Entrance Terminals with Flat Grades of Less Than 3 Percent~~

~~U.S. Customary~~

~~Acceleration Lane Length, L_a (ft) for Design Speed of Controlling Feature on Ramp, V' (mph)~~										
~~Highway~~		~~Stop Condition~~	~~15~~	~~20~~	~~25~~	~~30~~	~~35~~	~~40~~	~~45~~	~~50~~
~~Design Speed, V (mph)~~	~~Merge Speed, V_a (mph)~~	~~Average Running Speed (i.e., Initial Speed) at Controlling Feature on Ramp, V'_a (mph)~~								
		~~0~~	~~14~~	~~18~~	~~22~~	~~26~~	~~30~~	~~36~~	~~40~~	~~44~~
~~30~~	~~23~~	~~180~~	~~140~~	—	—	—	—	—	—	—
~~35~~	~~27~~	~~280~~	~~220~~	~~160~~	—	—	—	—	—	—
~~40~~	~~31~~	~~360~~	~~300~~	~~270~~	~~210~~	~~120~~	—	—	—	—
~~45~~	~~35~~	~~560~~	~~490~~	~~440~~	~~380~~	~~280~~	~~160~~	—	—	—
~~50~~	~~39~~	~~720~~	~~660~~	~~610~~	~~550~~	~~450~~	~~350~~	~~130~~	—	—
~~55~~	~~43~~	~~960~~	~~900~~	~~810~~	~~780~~	~~670~~	~~550~~	~~320~~	~~150~~	—
~~60~~	~~47~~	~~1200~~	~~1140~~	~~1100~~	~~1020~~	~~910~~	~~800~~	~~550~~	~~420~~	~~180~~
~~65~~	~~50~~	~~1410~~	~~1350~~	~~1310~~	~~1220~~	~~1120~~	~~1000~~	~~770~~	~~600~~	~~370~~
~~70~~	~~53~~	~~1620~~	~~1560~~	~~1520~~	~~1420~~	~~1350~~	~~1230~~	~~1000~~	~~820~~	~~580~~
~~75~~	~~55~~	~~1790~~	~~1730~~	~~1630~~	~~1580~~	~~1510~~	~~1420~~	~~1160~~	~~1040~~	~~780~~
~~80~~	~~57~~	~~2000~~	~~1900~~	~~1800~~	~~1750~~	~~1680~~	~~1600~~	~~1340~~	~~1240~~	~~980~~

~~Note: Uniform 50:1 to 70:1 tapers are recommended where lengths of acceleration lanes exceed 1,300 ft.~~

~~V = design speed of highway (mph)~~

~~V_a = merge speed (mph)~~

~~V' = design speed of controlling feature on ramp (mph)~~

~~V'_a = average running speed (i.e., initial speed) at controlling feature on ramp (mph)~~

~~L_a = acceleration lane length (ft)~~

~~Metric~~

~~Acceleration Lane Length, L_a (m) for Design Speed of Controlling Feature on Ramp, V' (km/h)~~									
~~Highway~~		~~Stop Condition~~	~~20~~	~~30~~	~~40~~	~~50~~	~~60~~	~~70~~	~~80~~
~~Design Speed, V (km/h)~~	~~Merge Speed, V_a (km/h)~~	~~Average Running Speed (i.e., Initial Speed) at Controlling Feature on Ramp, V'_a (km/h)~~							
		~~0~~	~~20~~	~~28~~	~~35~~	~~42~~	~~51~~	~~63~~	~~70~~
~~50~~	~~37~~	~~60~~	~~50~~	~~30~~	—	—	—	—	—
~~60~~	~~45~~	~~95~~	~~80~~	~~65~~	~~45~~	—	—	—	—
~~70~~	~~53~~	~~150~~	~~130~~	~~110~~	~~90~~	~~65~~	—	—	—
~~80~~	~~60~~	~~200~~	~~180~~	~~165~~	~~145~~	~~115~~	~~65~~	—	—
~~90~~	~~67~~	~~260~~	~~245~~	~~225~~	~~205~~	~~175~~	~~125~~	~~35~~	—
~~100~~	~~74~~	~~345~~	~~325~~	~~305~~	~~285~~	~~255~~	~~205~~	~~110~~	~~40~~
~~110~~	~~81~~	~~430~~	~~410~~	~~390~~	~~370~~	~~340~~	~~290~~	~~200~~	~~125~~
~~120~~	~~88~~	~~545~~	~~530~~	~~515~~	~~490~~	~~460~~	~~410~~	~~325~~	~~245~~
~~130~~	~~92~~	~~610~~	~~580~~	~~550~~	~~530~~	~~520~~	~~500~~	~~375~~	~~300~~

~~Note: Uniform 50:1 to 70:1 tapers are recommended where lengths of acceleration lanes exceed 400 m.~~

~~V = design speed of highway (km/h)~~

~~V_a = merge speed (km/h)~~

~~V' = design speed of controlling feature on ramp (km/h)~~

~~V'_a = average running speed (i.e., initial speed) at controlling feature on ramp (km/h)~~

~~L_a = acceleration lane length (m)~~

Source: *A Policy on Geometric Design of Highways and Streets* (2018) by AASHTO, Washington, D.C. Used with permission.

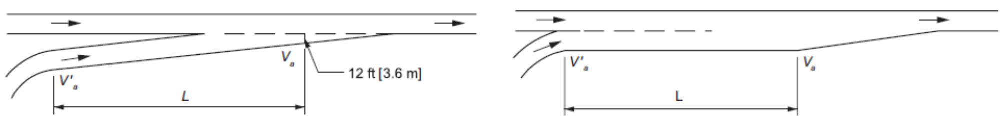
Proposed Table 10-4. Minimum Acceleration Lane Lengths for Entrance Terminals with Flat Grades of Less Than 3 Percent

U.S. Customary										
Highway		Acceleration Lane Length, L_a (ft) for Design Speed of Controlling Feature on Ramp, V' (mph)								
Design Speed, V (mph)	Merge Speed, V_a (mph)	Stop Condition	15	20	25	30	35	40	45	50
30	23	180	130	-	-	-	-	-	-	-
35	27	280	210	130	-	-	-	-	-	-
40	31	360	290	240	150	-	-	-	-	-
45	35	560	480	400	310	170	-	-	-	-
50	39	720	650	570	470	330	170	-	-	-
55	43	960	890	770	700	540	360	140	-	-
60	47	1200	1120	1060	940	780	600	370	130	-
65	50	1410	1340	1270	1130	980	800	580	320	-
70	53	1620	1540	1470	1330	1210	1020	800	530	200
75	55	1790	1710	1580	1490	1370	1200	960	730	380
80	57	2000	1880	1750	1660	1530	1380	1130	920	560

Note: Uniform 50:1 to 70:1 tapers are recommended where lengths of acceleration lanes exceed 1,300 ft.
V = design speed of highway (mph)
V_a = merge speed (mph)
V' = design speed of controlling feature on ramp (mph)
L_a = acceleration lane length (ft)

Metric									
Highway		Acceleration Lane Length, L_a (m) for Design Speed of Controlling Feature on Ramp, V' (km/h)							
Design Speed, V (km/h)	Merge Speed, V_a (km/h)	Stop Condition	20	30	40	50	60	70	80
50	37	60	50	25	-	-	-	-	-
60	45	95	80	60	25	-	-	-	-
70	53	150	130	105	70	20	-	-	-
80	60	200	180	160	125	70	-	-	-
90	67	260	245	220	185	130	60	-	-
100	74	345	325	300	260	205	135	45	-
110	81	430	410	385	345	290	220	130	20
120	88	545	530	510	465	405	335	245	120
130	92	610	580	545	505	465	415	300	175

Note: Uniform 50:1 to 70:1 tapers are recommended where lengths of acceleration lanes exceed 1,300 ft.
V = design speed of highway (km/h)
V_a = merge speed (km/h)
V' = design speed of controlling feature on ramp (km/h)
L_a = acceleration lane length (m)



Taper Type Parallel Type

Table 10-5. Speed Change Lane Adjustment Factors as a Function of Grade

U.S. Customary					
Design Speed of Highway (mph)	Deceleration Lanes				
	Ratio of Length on Grade to Length on Level for Design Speed of Turning Curve (mph)a				
All Speeds	3 to 4% upgrade 0.9				3 to 4% downgrade 1.2
All Speeds	5 to 6% upgrade 0.8				5 to 6% downgrade 1.35
Design Speed of Highway (mph)	Acceleration Lanes				
	Ratio of Length on Grade to Length on Level for Design Speed of Turning Curve (mph)a				
	20	30	40	50	All Speeds
3 to 4% Upgrade					3 to 4% Downgrade
40	1.3	1.3	—	—	0.7
45	1.3	1.35	—	—	0.675
50	1.3	1.4	1.4	—	0.65
55	1.35	1.45	1.45	—	0.625
60	1.4	1.5	1.5	1.6	0.6
65	1.45	1.55	1.6	1.7	0.6
70	1.5	1.6	1.7	1.8	0.6
75	1.6	1.7	1.8	2.0	0.6
80	1.7	1.8	2.0	2.1	0.6
5 to 6% Upgrade					5 to 6% Downgrade
40	1.5	1.5	—	—	0.6
45	1.5	1.6	—	—	0.575
50	1.5	1.7	1.9	—	0.55
55	1.6	1.8	2.05	—	0.525
60	1.7	1.9	2.2	2.5	0.5
65	1.85	2.05	2.4	2.75	0.5
70	2.0	2.2	2.6	3.0	0.5
75	2.15	2.35	2.8	3.25	0.5
80	2.3	2.5	3	3.5	0.5

[a] Ratio from this table multiplied by the length in Table 10-4 or Table 10-6 gives length of speed change lane on grade.

Metric						
Design Speed of Highway (km/h)	Deceleration Lanes					
	Ratio of Length on Grade to Length on Level for Design Speed of Turning Curve (km/h)a					
All Speeds	3 to 4% upgrade 0.9				3 to 4% downgrade 1.2	
All Speeds	5 to 6% upgrade 0.8				5 to 6% downgrade 1.35	
Design Speed of Highway (km/h)	Acceleration Lanes					
	Ratio of Length on Grade to Length on Level for Design Speed of Turning Curve (km/h)a					
	40	50	60	70	80	All Speeds
3 to 4% Upgrade						3 to 4% Downgrade
60	1.3	1.4	1.4	—	—	0.7
70	1.3	1.4	1.4	1.5	—	0.65
80	1.4	1.5	1.5	1.5	1.6	0.65
90	1.4	1.5	1.5	1.5	1.6	0.6
100	1.5	1.6	1.7	1.7	1.8	0.6
110	1.5	1.6	1.7	1.7	1.8	0.6
120	1.5	1.6	1.7	1.7	1.8	0.6
130	1.6	1.7	1.8	1.8	1.8	0.6
5 to 6% Upgrade						5 to 6% Downgrade
60	1.5	1.5	—	—	—	0.6
70	1.5	1.6	1.7	—	—	0.6
80	1.5	1.7	1.9	1.8	—	0.55
90	1.6	1.8	2.0	2.1	2.2	0.55
100	1.7	1.9	2.2	2.4	2.5	0.5
110	2.0	2.2	2.6	2.8	3.0	0.5
120	2.15	2.35	2.8	3.2	3.5	0.5
130	2.3	2.5	3.0	3.2	3.5	0.5

[a] Ratio from this table multiplied by the length in Table 10-4 or Table 10-6 gives length of speed change lane on grade.

Source: *A Policy on Geometric Design of Highways and Streets* (2018) by AASHTO, Washington, D.C. Used with permission.

10.9.6.6 Single-Lane Free-Flow Terminals, Exits

The design criteria provided for minimum deceleration lane lengths at exit terminals assume ~~there is no deceleration on the main line prior to exiting~~ that some deceleration occurs prior to departure from the mainline. ~~Although it is common that drivers start decelerating prior to leaving the main line, the designer should not rely on this in the design of the exit ramp.~~ ~~However, in some cases, it may be prudent for the designer to assume that all deceleration takes place in the speed-change lane when determining the minimum deceleration lane length.~~

10.9.6.6.1 Taper-Type Exits

The taper-type exit fits the direct path preferred by most drivers, permitting them to follow an easy path within the diverging area. The taper-type exit terminal beginning with an outer edge alignment break usually provides a clear indication of the point of departure from the through lane and has generally been found to operate smoothly on high-volume freeways. The divergence angle is usually between 2 and 5 degrees.

Studies of this type of terminal show that most vehicles leave the through lane at relatively high speeds, thereby reducing the potential for rear-end collisions as a result of deceleration on the through lane. The speed change can be achieved off the traveled way as the exiting vehicle moves along the taper onto the ramp proper. Figure 10-73A shows a typical design for a taper-type exit.

Vehicles should decelerate after clearing the through-traffic lane and before reaching the point limiting design speed for the ramp proper. The length available for deceleration may be assumed to extend from a point where the right edge of the tapered wedge is about 12 ft [3.6 m] from the right edge of the right through lane to the location of the controlling feature on the ramp. This feature may be the point of initial curvature (i.e., the first horizontal curve on the ramp), or it may be the crossroad terminal for a straight ramp. The length provided between these points should be at least as great as the distance needed to accomplish the appropriate deceleration, which is governed by the speed of traffic on the through lane and the speed to be attained on the ramp. Deceleration may end in a complete stop, as at a crossroad terminal for a diamond inter- change, or the critical speed may be governed by the curvature of the ramp roadway. The distance needed for deceleration in advance of the controlling feature is governed by the speed differential between the speed of vehicles departing the highway and the design speed (or advisory speed) on the controlling feature (e.g., horizontal curve) of the ramp. The use of curve design speed is a departure from prior AASHTO guidance, which utilized assumed values for average running speed at the controlling curve. Recent field studies showed that passenger vehicle speeds entering the controlling horizontal curve tended to exceed the prior AASHTO-assumed values for average running speeds at the curve. Minimum deceleration lengths for various combinations of design speeds for the highway and for the ramp roadway are given in Table 10-6. Grade adjustments are given in Table 10-5.

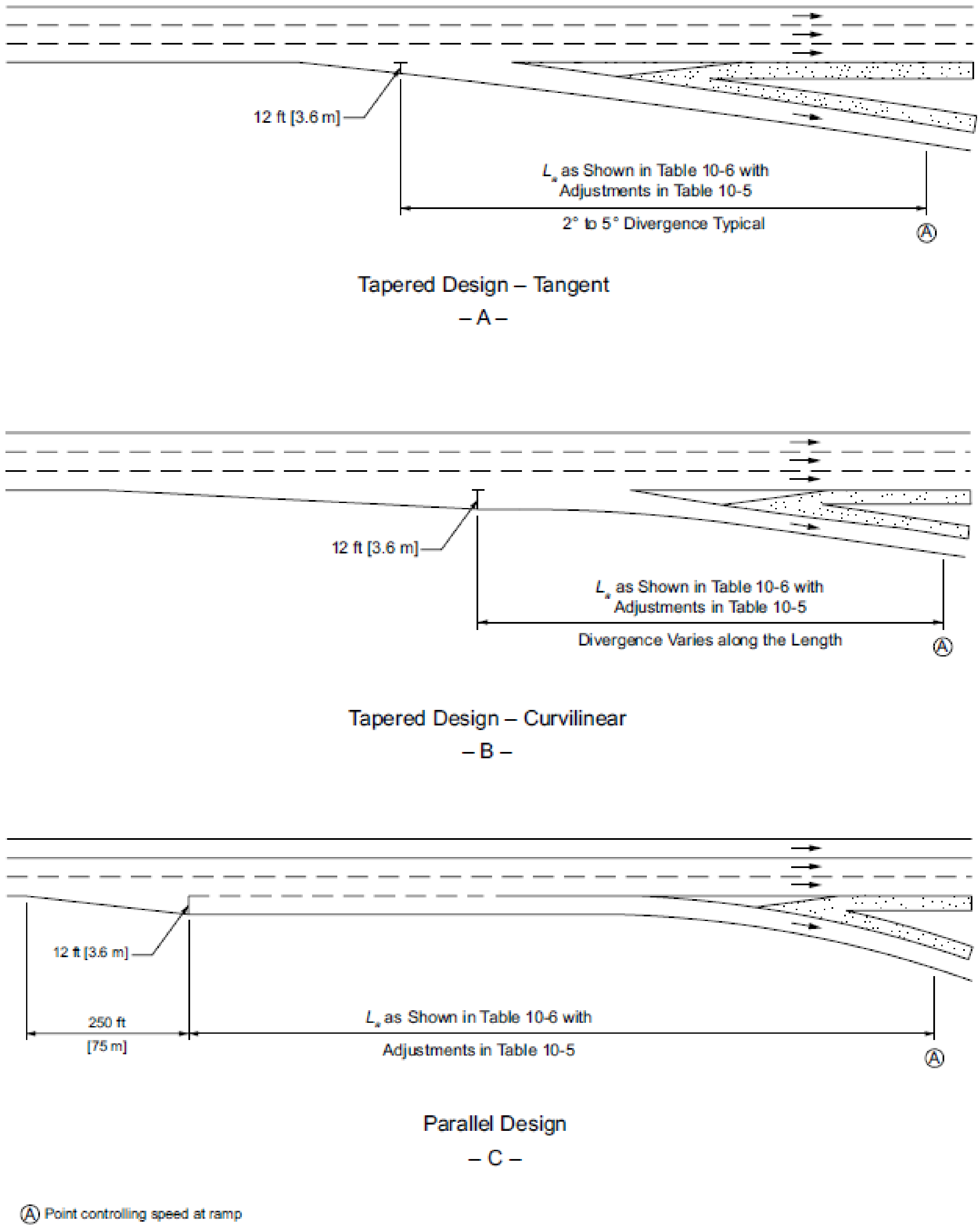
Although it is not desirable for vehicles to decelerate on the freeway main line prior to moving into a deceleration lane, recent research (*21, NCHRP 15-75*) has found that this does typically occur, as evidenced by lower diverge speeds compared to the adjacent mainline. ~~Because the values in Table 10-6 for minimum deceleration lane length on exit ramps do not account for any deceleration in the through lanes, these design values provide a conservative estimate for design.~~ However, in some cases, it may be prudent for the designer to assume that all deceleration takes place in the speed-change lane when determining the minimum deceleration lane length.

The taper-type exit terminal design can be used advantageously in developing the desired long, narrow, triangular emergency maneuver area just upstream from the exit nose located at a proper offset from both the through lane and separate ramp lane. The taper configuration also works well in the length-width superelevation adjustments to obtain a ramp cross slope different from that of the through lane.

The width of the recovery area or the distance between the inner edges of the diverging lanes at the ramp nose is usually 20 to 30 ft [6.0 to 9.0 m]. This entire area should be paved to provide a maneuver and recovery area, but the desired travel path for the ramp roadway should be clearly delineated by pavement markings.

10.9.6.6.2 Parallel-Type Exits

A parallel-type exit terminal usually begins with a taper, followed by an added lane that is parallel to the traveled way. A typical parallel-type exit terminal is shown in Figure 10-73C. This type of terminal provides an inviting exit area, because the foreshortened view of the taper and the added width are very apparent. A parallel-type exit operates best when drivers choose to exit the through lane sufficiently in advance of the exit nose to permit deceleration to occur on the added lane (deceleration lane) and allows them to follow a path similar to that encouraged by a taper design. Drivers who do not exit the through lane sufficiently in advance of the exit nose will likely utilize a more abrupt reverse-curve maneuver, which is somewhat unnatural and can sometimes result in the driver slowing in the through lane. In locations where both the main line and ramp carry high volumes of traffic, the deceleration lane provided by the parallel-type exit provides storage for vehicles that would otherwise undesirably queue up on the through lane or on a shoulder, if available.



Source: *A Policy on Geometric Design of Highways and Streets* (2018) by AASHTO, Washington, D.C. Used with permission.

Figure 10-73. Exit Ramps- Single Lane

The length of a parallel-type deceleration lane is usually measured from the point where the added lane attains a 12-ft [3.6-m] width to the point where the alignment of the ramp roadway departs from the alignment of the freeway. Where the ramp proper is curved, it is desirable to provide a transition at the end of the deceleration lane. A compound curve may be used with the initial curve desirably having a long radius of about 1,000 ft [300 m] or more. A transition or a long radius curve is also desirable if the deceleration lane connects with a relatively straight ramp. In such cases, a portion of the ramp may be considered as a part of the deceleration length, thus shortening to some extent the appropriate length of contiguous parallel lane. Minimum lengths are given in Table 10-6, and adjustments for grades are given in Table 10-5. Longer parallel-type deceleration lanes are more likely to be used properly by motorists than shorter lanes. ~~Lengths of at least 800 ft [240 m] are desirable.~~

Providing deceleration lanes longer than the minimum values listed in Table 10-6 may promote more casual deceleration by exiting drivers, particularly under uncongested or lightly congested conditions. This is not necessarily a negative result, but it may change the operational characteristics of the ramp, as those drivers may maintain higher speeds further into the speed-change lane and possibly into the ramp proper.

The taper portion of a parallel-type deceleration lane should have a taper of approximately 15:1 to 25:1 [longitudinal:transverse]. A long taper indicates the general path to be followed and reduces the unused portion of the deceleration lane. However, a long taper tends to entice the through driver into the deceleration lane. A short taper produces a better “target” to the approaching driver, giving a positive indication of the added lane ahead.

~~***Table 10-6. Minimum Deceleration Lane Lengths for Exit Terminals with Flat Grades of Less Than 3 Percent***~~

~~**U.S. Customary**~~										
~~Deceleration Lane Length, L_a (ft) for Design Speed of Controlling Feature on Ramp, V' (mph)~~										
~~Highway Design Speed, V (mph)~~	~~Diverge Speed, V_a (mph)~~	~~Stop Condition~~	~~15~~	~~20~~	~~25~~	~~30~~	~~35~~	~~40~~	~~45~~	~~50~~
		~~Average Running Speed at Controlling Feature on Ramp, V'_a (mph)~~								
		~~0~~	~~14~~	~~18~~	~~22~~	~~26~~	~~30~~	~~36~~	~~40~~	~~44~~
~~30~~	~~28~~	~~235~~	~~200~~	~~170~~	~~140~~	—	—	—	—	—
~~35~~	~~32~~	~~280~~	~~250~~	~~210~~	~~185~~	~~150~~	—	—	—	—
~~40~~	~~36~~	~~320~~	~~295~~	~~265~~	~~235~~	~~185~~	~~155~~	—	—	—
~~45~~	~~40~~	~~385~~	~~350~~	~~325~~	~~295~~	~~250~~	~~220~~	—	—	—
~~50~~	~~44~~	~~435~~	~~405~~	~~385~~	~~355~~	~~315~~	~~285~~	~~225~~	~~175~~	—
~~55~~	~~48~~	~~480~~	~~455~~	~~440~~	~~410~~	~~380~~	~~350~~	~~285~~	~~235~~	—
~~60~~	~~52~~	~~530~~	~~500~~	~~480~~	~~460~~	~~430~~	~~405~~	~~350~~	~~300~~	~~240~~
~~65~~	~~55~~	~~570~~	~~540~~	~~520~~	~~500~~	~~470~~	~~440~~	~~390~~	~~340~~	~~280~~
~~70~~	~~58~~	~~615~~	~~590~~	~~570~~	~~550~~	~~520~~	~~490~~	~~440~~	~~390~~	~~340~~
~~75~~	~~61~~	~~660~~	~~635~~	~~620~~	~~600~~	~~575~~	~~535~~	~~490~~	~~440~~	~~390~~
~~80~~	~~64~~	~~705~~	~~680~~	~~665~~	~~645~~	~~620~~	~~580~~	~~535~~	~~490~~	~~440~~

~~V = design speed of highway (mph)~~

~~V_a = average running speed on highway (i.e., diverge speed) (mph)~~

~~V' = design speed of controlling feature on ramp (mph)~~

~~V'_a = average running speed at controlling feature on ramp (mph)~~

~~L_a = deceleration lane length (ft)~~

~~**Metric**~~									
~~Deceleration Lane Length, L_a (m) for Design Speed of Controlling Feature on Ramp, V' (km/h)~~									
~~Highway Design Speed, V (km/h)~~	~~Diverge Speed, V_a (km/h)~~	~~Stop Condition~~	~~20~~	~~30~~	~~40~~	~~50~~	~~60~~	~~70~~	~~80~~
		~~Average Running Speed at Controlling Feature on Ramp, V'_a (km/h)~~							
		~~0~~	~~20~~	~~28~~	~~35~~	~~42~~	~~51~~	~~63~~	~~70~~
~~50~~	~~47~~	~~75~~	~~70~~	~~60~~	~~45~~	—	—	—	—
~~60~~	~~55~~	~~95~~	~~90~~	~~80~~	~~65~~	~~55~~	—	—	—
~~70~~	~~63~~	~~110~~	~~105~~	~~95~~	~~85~~	~~70~~	~~55~~	—	—
~~80~~	~~70~~	~~130~~	~~125~~	~~115~~	~~100~~	~~90~~	~~80~~	~~55~~	—
~~90~~	~~77~~	~~145~~	~~140~~	~~135~~	~~120~~	~~110~~	~~100~~	~~75~~	~~60~~
~~100~~	~~85~~	~~170~~	~~165~~	~~155~~	~~145~~	~~135~~	~~120~~	~~100~~	~~85~~
~~110~~	~~91~~	~~180~~	~~180~~	~~170~~	~~160~~	~~150~~	~~140~~	~~120~~	~~105~~
~~120~~	~~98~~	~~200~~	~~195~~	~~185~~	~~175~~	~~170~~	~~155~~	~~140~~	~~120~~
~~130~~	~~103~~	~~215~~	~~210~~	~~205~~	~~195~~	~~185~~	~~170~~	~~155~~	~~135~~

~~V = design speed of highway (km/h)~~

~~V_a = average running speed on highway (i.e., diverge speed) (km/h)~~

~~V' = design speed of controlling feature on ramp (km/h)~~

~~V'_a = average running speed at controlling feature on ramp (km/h)~~

~~L_a = deceleration lane length (m)~~

Source: *A Policy on Geometric Design of Highways and Streets* (2018) by AASHTO, Washington, D.C. Used with permission.

Proposed Table 10-6. Minimum Deceleration Lane Lengths for Exit Terminals with Flat Grades of Less Than 3 Percent

U.S. Customary

Highway Design Speed, V (mph)	Diverge Speed, V_a (mph)	Deceleration Lane Length, L_a (ft) for Design Speed of Controlling Feature on Ramp, V' (mph)								
		Stop Condition	15	20	25	30	35	40	45	50
30	28	235	195	155	125	-	-	-	-	-
35	32	280	245	195	160	125	-	-	-	-
40	36	320	290	250	210	145	75	-	-	-
45	40	385	345	310	275	215	165	-	-	-
50	44	435	400	370	335	285	230	170	-	-
55	48	480	450	430	390	350	305	240	175	-
60	52	530	495	470	440	400	355	305	240	205
65	55	570	535	510	480	440	395	345	280	215
70	58	615	585	560	530	490	445	395	330	265
75	61	660	630	610	580	545	490	445	380	320
80	64	705	675	655	625	590	535	495	430	370

V = design speed of highway (mph)
V_a = average running speed on highway (i.e., diverge speed) (mph)
V' = design speed of controlling feature on ramp (mph)
L_a = deceleration lane length (ft)

Metric

Highway Design Speed, V (km/h)	Diverge Speed, V_a (km/h)	Deceleration Lane Length, L_a (m) for Design Speed of Controlling Feature on Ramp, V' (km/h)							
		Stop Condition	20	30	40	50	60	70	80
50	47	75	70	60	40	-	-	-	-
60	55	95	90	80	60	45	-	-	-
70	63	110	110	95	80	60	35	-	-
80	70	130	130	115	95	80	65	40	-
90	77	145	145	135	115	100	85	60	40
100	85	170	165	155	140	125	105	85	75
110	91	180	185	170	155	140	125	105	85
120	98	200	195	185	170	160	140	125	100
130	103	215	215	205	190	175	155	140	115

V = design speed of highway (km/h)
V_a = average running speed on highway (i.e., diverge speed) (km/h)
V' = design speed of controlling feature on ramp (km/h)
L_a = deceleration lane length (m)